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HYDRAULIC MODELING TO INFORM STREAM MAINTENANCE PRACTICES IN SOUTH CAROLINA

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HYDRAULIC MODELING TO INFORM STREAM MAINTENANCE STRATEGIES IN SOUTH CAROLINA

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by
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ABSTRACT

The Crabtree Canal located in Horry County, SC is the main conveyor of stormwater in a watershed that has undergone considerable urbanization in the last few decades. Evidence of bank instability and inchannel erosion is widely seen in the Crabtree Canal system. Principal sediment inputs are landscape sources and in-channel sources. A study was initiated to provide a working management tool to determine hydrodynamic conditions on the watershed driven by a hypothetical storm event and alternative channel configurations. The management tool comprised of a one-dimensional HEC-RAS model of the Crabtree Canal and was developed to aid the Horry County stormwater department in determining potential zones of stream instability and in evaluating alternate stream management techniques. Average velocity, hydraulic depth and shear stress were used to quantify changes in flow regime. Alternative stream management techniques included different floodplain configurations being implemented on the existing geometry of the channel. The management tool modeled average velocity, hydraulic depth, and shear stress decreasing as floodplain width increased relative to the top width of the main channel. The model also suggested that potential points of stream instability in the system were located at points of inflection in the stream bed profile and at points where the bed profile transitioned to a steeper slope.

DEDICATION

This thesis is dedicated to Debbie Owens, Randy Owens and Juanita Norton; Mama for providing strength, comfort and wisdom during the tough times, Daddy for setting the best example of hard work and honesty that one could have, Grandma for sharing everything she had with me and for never complaining about me cluttering up her dining room table when I used it for a study desk.

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– Charles V. Privette III, PhD, PE

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CHAPTER ONE

INTRODUCTION

Urbanization of a watershed can cause considerable changes in stream flow regime due to increased imperviousness and decreased natural area, alteration of the drainage network, and changes to channel morphology (Hammer 1972). Urban development in the form of shopping centers, parking lots, roads, and houses increases the percentage of impervious area in a watershed which in turn increases the volume of runoff and the magnitude of peak flows at the watershed outlet. Stream channels tend to remain in a state of quasi-equilibrium with the current flow and sediment regime until outside forces, whether natural or anthropogenic, impose instabilities on the system (Hammer, 1972). Increased flow volumes, velocities, and higher peaks induced by urbanization of the watershed tend to result in the enlargement of the stream by either incision or widening to accommodate larger flows. In many cases, this natural enlargement of the stream channel is brought about by human influences, and flooding or damage to property becomes a serious concern (Hammer, 1972; Neller, 1989). Crabtree Canal is a stream channel that serves as the principal conduit for stormwater flows in the city of Conway, SC. Crabtree Canal was modified by the US Army Corps of Engineers in the early 1960's in response to flooding issues in the city.

The watershed is approximately 70 km² (27 mi²) at its confluence with the Kingston Lake Swamp drainage network (Figure 1.1). 18% of the land is developed, 25% of the land is forested, 31% of the land is pasture or cultivated crops and 26% of the land is classified as wetlands (MRLC, 2009). The dominant soil type present in the study site are Meggett loams and Wahee fine sandy loams, these are poorly draining soils characterized as hydrologic type D soils. In all, hydrologic group D soils cover approximately 54% of the watershed, while 28% of the soils were type C soils, 11 % of the soils were type B soils and 7% of the soils were type A soils (USDA-NRCS, 2008).

The downstream reaches of Crabtree Canal are tidally influenced and are also affected by backwater effects from the much larger Waccammaw River into which Kingston Lake Swamp flows just 3.7 km downstream of its confluence with Crabtree Canal (Figure 1.1). In order to remedy urban flooding problems in Conway, the US Army Corps of Engineers straightened and reshaped the channel to a large trapezoidal shape. These channel modifications disconnected the channel from its natural floodplain. The excavated soil was piled up along the channel and further disconnected the floodplain from the main channel. Crabtree Canal currently exhibits characteristics of a Rosgen Type F or G channel (Rosgen, 1994).

Due to the increase in sediments and sediment deposition, periodic channel dredging was carried out to maintain the ability of the channel to convey the increased stormwater discharges. These periodic dredgings reshaped the channel and removed any rooted vegetation along the boundaries of the channel. Principal sediment inputs are likely from landscape sources and in-channel sources; however, a more accurate estimate of sediment sources is lacking. Evidence of bank instability and in channel erosion is widely seen in the Crabtree Canal system. The objectives of this study were:

- Develop a working management tool to determine hydrodynamic conditions on the watershed driven by a hypothetical storm event and alternative channel geometry configurations.
- Quantify the relative performance of different floodplain configurations using the management tool,
- Identify possible zones of instability in Crabtree Canal
- Identify the most suitable locations and size for floodplain alteration.

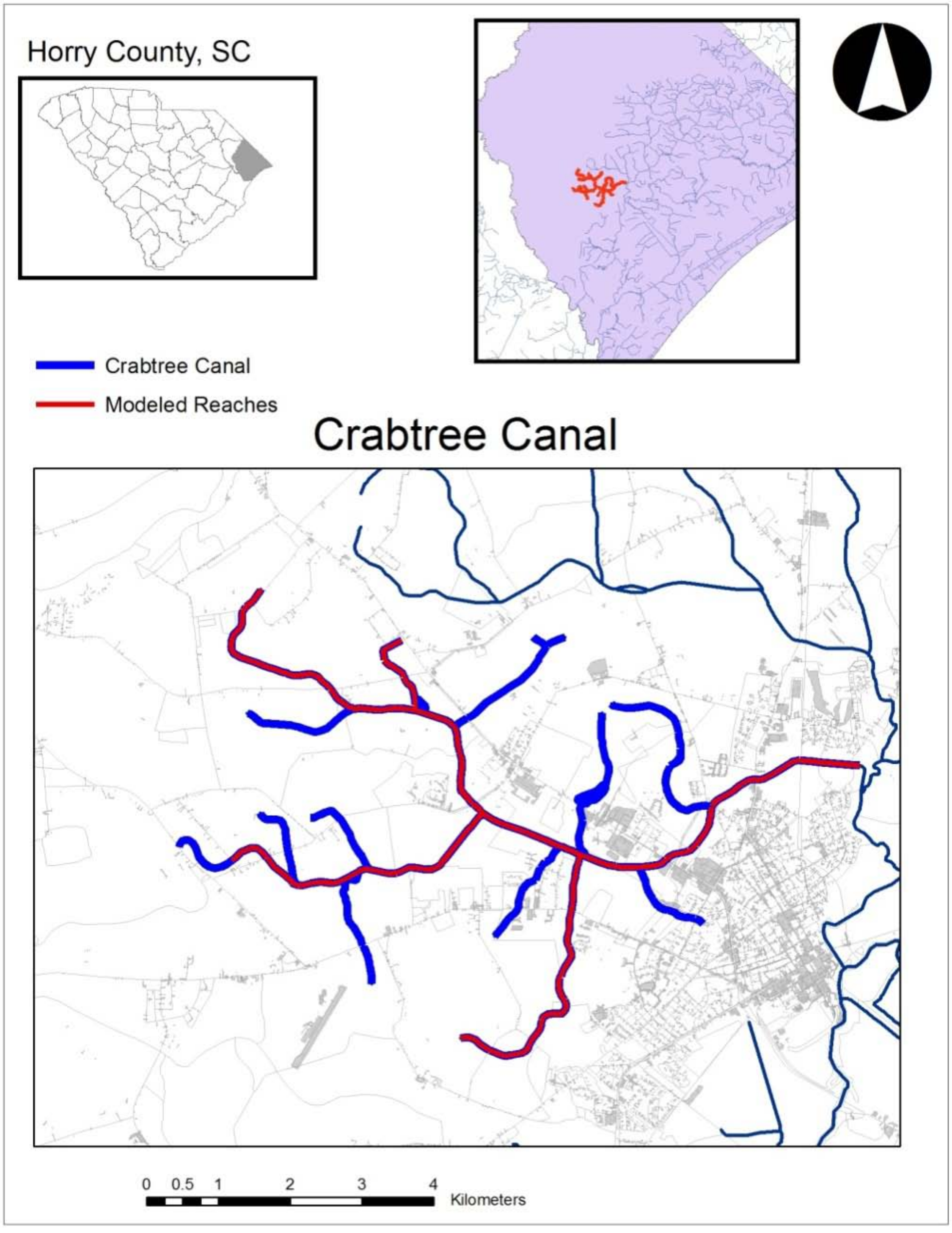


Figure 1.1. Geographical location of study site.

CHAPTER TWO

BACKGROUND

Traditional maintenance practice versus two-stage channel design

Traditional practices to facilitate urban stormwater runoff typically involve the straightening and channelizing of natural drainage systems into deep, narrow trapezoidal shaped channels. This type of channel reshaping disconnects flows from the natural floodplain a discontinuity that is sometimes exacerbated when the excavated soils are piled along the channel margins. Evidence of this practice is visible along much of Crabtree Canal. In order to maintain the ability to convey water efficiently, channels of this nature must periodically be dredged to remove built up sediments from the bottom of the channel. The initial channel reshaping and then the sequential dredging can cause continual erosion to occur within the channel due to instabilities and exposure of the bank due to this earthwork (Neller, 1989).

The straightening of a stream shortens the downvalley component of stream length that results in an increase in average bed slope. A steeper bed slope (S), results in increased stream power (Ω), bed shear stresses (τ) and average flow velocities (V). Worthy (2005) defines stream power as the energy available to transport sediment and gives the following equation for stream power per unit length of channel:

$$\Omega = \gamma QS \qquad (1)$$

Where: Ω= stream power per unit length (W/m),

γ= specific weight of water (9810 N/m^3),

Q= volume flow rate (m^3/s) and

S=slope (m/m).

Shear stress is found using the following equation:

$$\tau = \gamma RS \tag{2}$$

Where: τ= average shear stress (N/m^2) and

R= hydraulic radius (m) (Haan et al. 1994).

A steeper slope affects the average velocity in a channel following Robert Manning's well known equation (Haan et al. 1994):

$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{3}$$

Where: V= average velocity (m/s) and

n= roughness coefficient known as Manning's n (dimensionless).

If increased discharges are not allowed to flow out onto an active floodplain, the increased stream power associated with these flows could cause the channel to incise and/or widen. Downcutting and widening of a channel could cause a positive feedback mechanism as more flows are confined within the main channel resulting in greater shear stresses on channel bed and banks (Ward et al., 2008).

This study explores the implications and the results of modifying the geometry of Crabtree Canal from a single stage system into a two-stage channel design with a low flow main channel (stage one) and an active floodplain (stage two) (see Figure 2.1). Focus is placed on the width and overall geometry of the floodplain relative to the main channel. Ward et al. (2008) use the term channel–forming discharge to describe both the bankfull and effective discharge of a channel in their analysis of the requirements needed to sustain active floodplains. Bankfull discharge is flow that fills the channel and begins to spill over onto the floodplain. Effective discharge defined as the mean of the arithmetic discharge increment that transports the largest fraction of annual sediment load over a period of years (Andrews, 1980). In a two-stage design, the main channel is sized large enough to accommodate the channel forming discharge of the system. The typical return period for a system's effective discharge is 1.5 years (Haan et al., 1994). However, there is still much debate over the recurrence interval associated with the effective discharge. Some studies suggest more frequent return periods such as less than 1.5 years (Crowder and Knapp, 2004; Powell et al., 2006) while some studies propose a channel-forming discharge recurrence interval of up to five years (Petit and Pauquet, 2004).

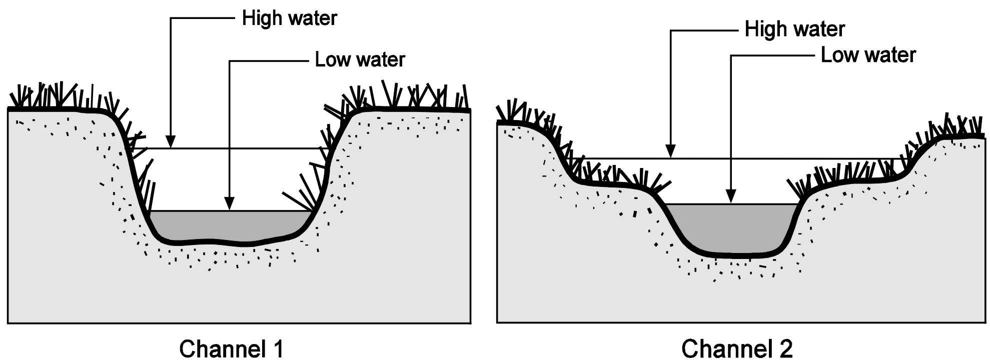


Figure 2.1. A single stage channel on the left is contrasted with a two-stage channel on the right *(Figure source: The Ohio State University Extension Service)*

The role of an active floodplain is to provide excess flow area for flows that overtop the banks of a channel. A floodplain dissipates excess stream power of increased discharge rates by increasing the flow area and decreasing the flow velocity. Due to increased vegetation, roughness resistances are normally greater on a floodplain. The lower flow velocities associated with a floodplain allow more suspended sediments to fall out of suspension. In order to relate the width of the main channel to the width of the floodplain, an expression known as floodplain ratio (FPR) is often used. A FPR for the purposes of this study is defined as the ratio of the floodplain width at the bottom of the floodplain to the width of the main channel at the top of the main channel (Figure 2.2). A study performed by Ward et al. (2008) concluded that a FPR between 5 and 10 is needed to obtain a self-sustaining system, but FPR smaller than 5 would still be of some benefit in terms of the stability of the system. Ward et al. (2008) examined the implications of increased floodplain width by carrying out analyses on a single hypothetical cross section with varying FPR values. This study seeks to expand on that study by looking at selected reaches of an actual drainage network, based on physical measurements of stream pattern, profile and dimension.

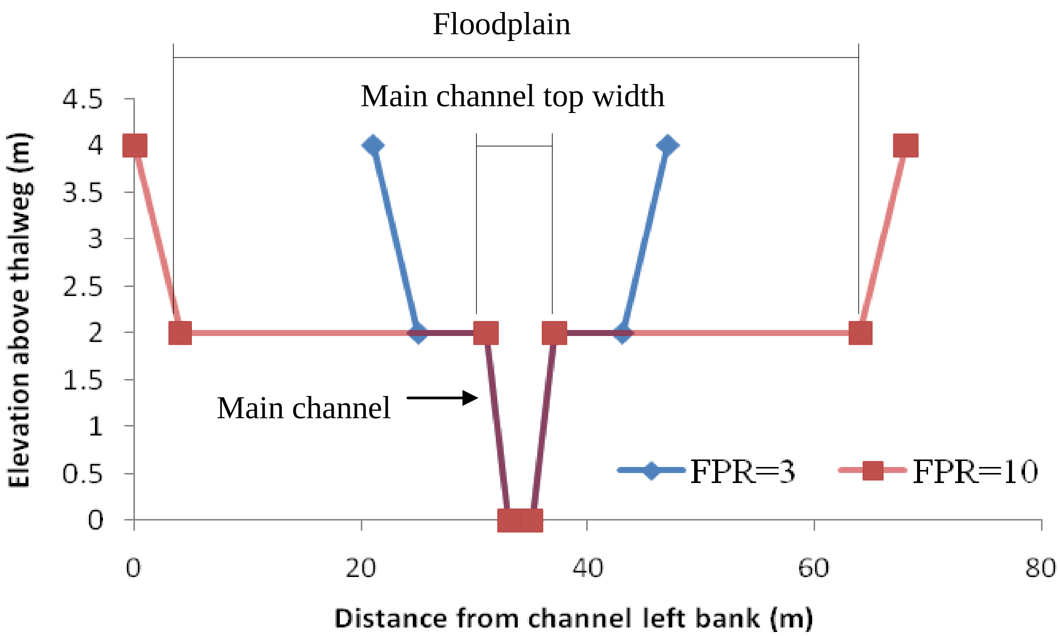


Figure 2.2. Schematic showing channel configuration at floodplain ratios of 3 and 10.

Stream naturalization/restoration

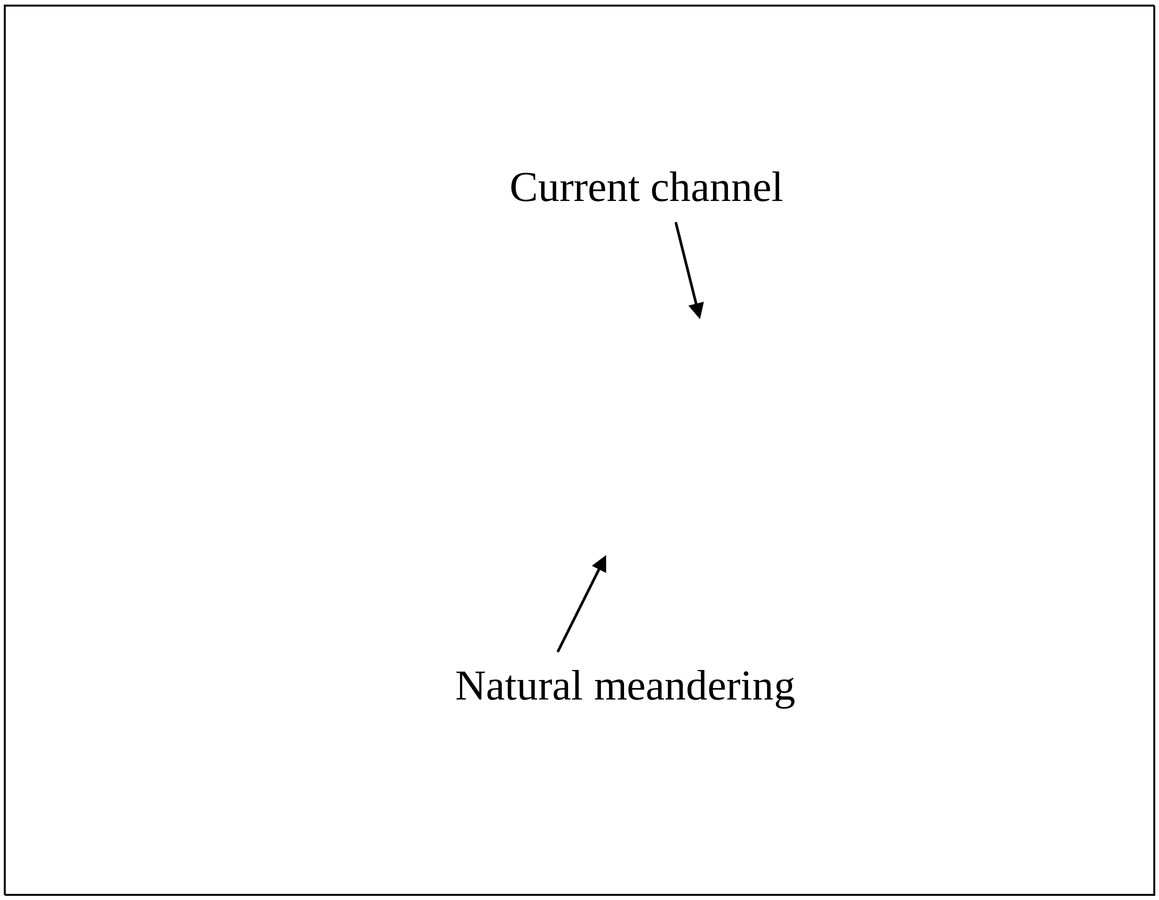
According to Levell and Chang (2008), streams are one of the most sensitive components of the landscape to disturbance. Activities such as dam construction, urban development and channelization can have a significant impact on nearby riparian systems by changing discharge rates and by altering sediment loads. Such impacts can eventually push a stream into disequilibrium, causing an alteration in the stream's morphology and riparian ecology. Channels in equilibrium are relatively stable in their morphology, with stable banks and bedforms (Levell and Chang, 2008). Some of the most affected reaches comprise headwater streams within agricultural and urban landscapes. In addition to the initial channelization of a stream, routine maintenance activities such as dredging also contribute to the degradation of a stream (Rhoads et al. 1999).

In recent decades, there has been a cultural shift toward a more responsible stewardship of the environment. The Natural Resources Conservation Service (NRCS) has recently put out an extensive handbook for stream restoration design (USDA-NRCS, 2007). Restoring streams to their natural condition is a popular approach to mitigating stream degradation (Palmer and Bernhardt, 2006). The National Research Council (1992) defines restoration as the complete structural and functional return of a biophysical system to a predisturbed state. According to Rhoads et al. (1999), complete restoration of agricultural drainage systems is unlikely for several reasons; the first reason being the lack of information on natural systems before they were disturbed. In order to return a system to pre-disturbed conditions, one must have detailed information of the stream's morphological and ecological configurations. Most alterations in agricultural and urban drainage systems occurred prior to the collection of such detailed environmental information. The second reason is the impact that development has caused is not limited to the stream itself, but often times to an entire watershed. Land cover and land use over an entire watershed plays a role in the morphology of a stream and therefore if the surrounding land is altered, it causes alterations to the receiving body of water. With large human populations today and great demand placed on land to produce agricultural products, it is highly unlikely that a total watershed can be returned to pristine state. Often times in agricultural communities, predisturbed conditions have little or no value to sustainable, community-based approaches to stream management (Rhoads et al, 1999).

An alternative to total restoration is the concept of stream naturalization (Rhoads et al, 1999). Stream naturalization alters an impacted stream so that it is in a more natural state. Naturalization defines a viable management goal for watersheds situated in landscapes characterized by intensive human modification of the biophysical environment (Rhoads et al., 1999). An understanding of the stream's geomorphic processes is essential to proactive efforts to bring about the naturalization of a stream (Levell and Chang, 2008). Relatively little work has been done to research the geometric designs of restored stream channels, active floodplains and riparian zones (Morris, 1996; Ward et al., 2008). Despite this fact, the foundation for stream restoration design has been researched in the form of alluvial channel regime-theory and hydraulic geometry (Hammer, 1972; Richards, 1982; Jackson and Van Haveren, 1984; Neller, 1989; Morris, 1996; Ward et al., 2008). Readily available hydraulic models, such as HEC-RAS, offer the capacity to model some of the implications of restoration efforts with sufficient accuracy. If models cannot be used to accurately simulate a restoration design, they can at least give insight into some of the complexity that govern such systems and some of the design criteria that need to be addressed. Design criteria may include the longevity, feasibility or cost of restoration.

Aerial photographs of Crabtree Canal show meanders in the stream before it was channelized and straightened to accommodate for higher peak flow values (Figure 2.3). Efforts were not made to remeander the channel, only to reconfigure the main channel to include a low flow channel and floodplain. This project is a part of ongoing research across the country involving stream restoration designs and their effectiveness in enhancing aquatic and terrestrial habitat, controlling sedimentation and erosion, and enhancing the overall aesthetics of waterways; such research includes but is not limited to that of Levell and Chang (2008) and Feyrer et al. (2006).

Figure 2.3. Natural meanders in stream before straightening.



Several different types of plant communities such as wetland forests, submergent aquatic vegetation and moist soil flora take root and thrive in active floodplain areas. Roots from these plants take hold and stabilize deposited sediments. Stabilization of these sediments prevents resuspension and improves water quality by reducing turbidity (Ahn et al. 2006). Flora in the floodplain also makes a significant contribution to the nutrient cycle in the aquatic ecosystem. The germination cycle of plants provide seeds, rhizomes and tubers as a source of food for local wildlife. The plant itself can also be a food source for local wildlife. When plants die or shed their leaves, the dead plant matter decomposes and contributes to organic matter in detritus-based food webs (Ahn et al., 2006). Floodplains provide fish and other aquatic fauna with refuge from high flows, a suitable place to spawn, and a suitable nursery habitat (Feyrer, 2006). Along with ecological benefits, the provision of additional floodplain, if implemented with this goal in mind, can provide the local citizenry with a suitable place to retreat from urbanization. Park plans can be incorporated into stream naturalization plans to provide people with a place to hunt, fish, hike, bike, or simply sit and enjoy the environment (Searns, 1995).

Urbanization can have a significant impact on a stream's water quality. Water quality degradation can come from both point and nonpoint source pollution (Moscrip and Montgomery, 1997). Projects designed to restore or maintain the inherent complexities of stream corridors, ecological linkages, and their physical connections are one solution to arrest the decline of aquatic and riparian species and to improve the Nation's water quality (USDA-NRCS, 2007). In the case of this study, vegetation covering the modified floodplain supplies top sediment with organic materials and oxygen resulting in the development of a rhizosphere that serves as habitat for microbes and other fauna (Vance et al., 1994). A denitrifying zone develops beneath the aerobic zone due to depletion of oxygen in this region (Chung et al., 2004). Both organic matter and nitrogen are removed when contaminated water infiltrates through the aerobic and denitrifying zones (Chung et al., 2004). Water exchange between river channels and unconfined aquifers in natural systems such as floodplains is now generally accepted as important sinks for organic matter and nitrogen through the biogeochemical processes in subsurface groundwater (Haycock and Burt, 1993; Tsushima et al., 2002).

Computer based hydrodynamic modeling

With advancements in technology and the availability of large empirical databases, there has been a greater availability of sophisticated numerical models. Haan et al. (1994) define a hydrologic model as "*a collection of physical laws and empirical observations written in mathematical terms and combined in such a way as to produce hydrologic estimates (outputs) based on a set of known and/or assumed conditions (inputs).*" The most common use of such models is to evaluate impact of some physical change on the system being modeled. Once a model has been developed and calibrated, various combinations of storage, channel modifications, land use changes and stream stabilizations can be more easily evaluated. A model is a very helpful tool to engineers and scientists; however, detailed knowledge of the system being modeled is still essential. The model does not adequately replace system knowledge; it just carries out mathematical computations (Haan et al., 1994).

Abad et al. (2008) define river morphodynamics as the interaction between hydrodynamics, sediment transport, bank erosion and bed morphology. Hydrodynamics and morphodynamics of a river or natural stream are highly complex interactions involving secondary flows, turbulent flows, sediment transport and channel migration. These highly complex interactions can result in migration of the stream or river, degradation of the bed, evolution of bedforms and variation in suspended sediment loads (Abad et al., 2008). It is possible to model such interactions using a complex 3D model. 3D models often require sophisticated implementation of boundary conditions (Ingham and Ma, 2005; Sotiropoulos, 2005) and are only applicable to reach scale domain cases (Abad et al., 2008). These problems can often be overcome with the use of either a 2D depth averaged model or a cross section averaged 1D model (Abad et al., 2008). In a study that included modeling flooding caused by glacial outburst, the computation time required for a 1D model was 2-5 minutes, whereas the 2D model required 24-36 hours for the same simulation (Alho, 2008). According to Hunter et al. (2008), there are five main representative classes of 2D that are applicable to urban hydraulic modeling: (1) implicit finite-difference solutions of the full 2D shallow water equations, (2) explicit finite-difference solutions of the full 2D shallow water equations, (3) explicit finite-volume solutions of the full 2D shallow water equations, (4) explicit finite-difference solutions to the 2D diffusion wave equations and (5) explicit analytical approximations to the 2D diffusion wave equations. All of the models used in the study by Hunter et al. (2008) utilized either the 2D shallow water equations or the 2D diffusion wave equations.

In this study a computer based modeling approach was used to simulate the effects of channel alteration on the hydrodynamics of a drainage network. The objective was to provide local stormwater agents with a tool to evaluate potential management strategies, suitable locations for intervention, and the effect on the overall conveyance of the drainage network. The scale of the study area, the evaluation of multiple channel configurations, and the desire of a relatively easy to use tool, necessitated the choice of a one-dimensional (1D) hydrodynamic model. The majority of people using this model (stormwater agents) would not have access to a computer with the ability to run models of a higher dimension. For this study a 1D model called HEC RAS was used to evaluate potential channel reconfiguration scenarios (www.hec.usace.army.mil/software/hec-ras/). The program is made available to the public by the US Army Corps of Engineers. HEC RAS is coded to solve the full 1D St. Venant equations. The computational procedure for establishing water surface elevations involves solving the 1D energy equation with frictional energy losses calculated using Manning's equation (Equation 3) (Morris, 1996). Manning's open channel equation is an empirical equation that is often used to determine average stream velocity in hydraulic models. Recent work by Gioia and Bombardelli (2002) suggest that Manning's equation has a theoretical basis based on turbulence theory. Due to the difference in the way that 1D and 2D models parameterize friction losses, the two types of models show different sensitivity levels to changes in Manning's n. Manning's n is often used to calibrate model output (Horritt and Bates, 2002; Yu and Lane, 2006a; Tayefi et al., 2007). 1D models are more sensitive to changes in Manning's n values (Horritt and Bates, 2001b; Yu and Lane, 2006a; Tayefi et al., 2007). Horritt and

Bates (2002) points out that 1D and 2D models utilize different process inclusions; the friction value has a different physical meaning and is drawn from a different distribution. Different responses to roughness values indicate that for some or all models, friction parameters are being used to compensate for different processes representations; thus the friction coefficient for a 1D model cannot be absolutely compared to that for a 2D model (Horritt and Bates, 2002).

CHAPTER THREE

METHODOLOGY

A United Stated Geological Survey (USGS) real time gage is located on the Crabtree Canal system at its crossing with Long Avenue. The gage coordinates are latitude 33°51’39”, longitude 79°02’29” with the hydrologic unit being 03040206 (USGS, 2009). The drainage area is 46 km^2 and the datum of the gage is 3 m (USGS, 2009). The gage is located about 0.5 km upstream from the confluence of Crabtree Canal and the Kingston Lake Swamp drainage network. The location of the gage was chosen as the downstream extent of the physical domain that was to be modeled. The upstream extent in both physical and computational domains was at Four Mile Road (see Figure 3.2), approximately 9,500 m (31,000 ft) upstream of the Long Avenue Bridge. Three tributaries were also modeled. Figure 3.1 provides the extent of the modeled reaches in the physical domain.

Within a Geographic Information System (GIS) environment, Light Detection and Ranging (LiDAR) data was used to develop a digital terrain model (DTM) both as a Trianguated Irregular Network (TIN) and as a raster format. The DTM was applied to the watershed and the surrounding area. A stream centerline layer was then added. The centerline was broken up by reaches and tributaries in the drainage network. Centerlines were drawn from upstream to downstream. Junction points connected each tributary to the main stem. After establishing the locations of the centerlines, each reach and tributary was assigned a unique reach and river name. Cross sections were extracted from the DTM at regular intervals along the length of the modeled system. These cross sections were only representative of the land above the water surface as LiDAR does not penetrate below the water surface. Manual topographic surveys were performed to determine below-water channel morphology.

Two different types of surveys were done. On both surveys a laser level was used to take an elevation reading at a point that was easily distinguishable on the LiDAR dataset such as a bridge deck or other high point. One method used an elevation reading to the water surface and then took depth readings to determine the bottom of channel geometry, while the other directly measured the entire channel cross section using a laser level set up at a known elevation. Water depth readings were taken at several points along the channel bed using a simple measuring rod. The geographic position of each depth reading was recorded with a hand held Trimble®[1] GPS unit as they were taken. Channel bed elevation readings were taken at the Long Ave bridge crossing, the Sherwood Rd bridge crossing, the railroad bridge crossing, the Hwy 701 bridge crossing, a point approximately midway between Highway 701 and the railroad, and the Oak Street bridge crossing (see Figure 3.1 for site for bridge locations). Topographic surveys were conducted on head water reaches to determine general channel shape and profile. Elevations measured along the channel thalweg helped determine the approximate slope of the channel. The latter survey method was used in the upstream sections of the system where the water depth was shallower. Channel depth readings taken from the surveys were interpolated to approximate channel bottom geometry along the entire channel.

[1] 2005 Trimble® GeoXT

Cross sections previously taken from the LiDAR data were altered in HEC RAS to include the bottom of the channel as well as the terrain above the water surface.

A total of 12 bridges and culverts were included in the model (Figure 3.1.) Dimensions of the modeled bridges were obtained from the Horry County office of the South Carolina Department of Transportation (Patrick, 2008).

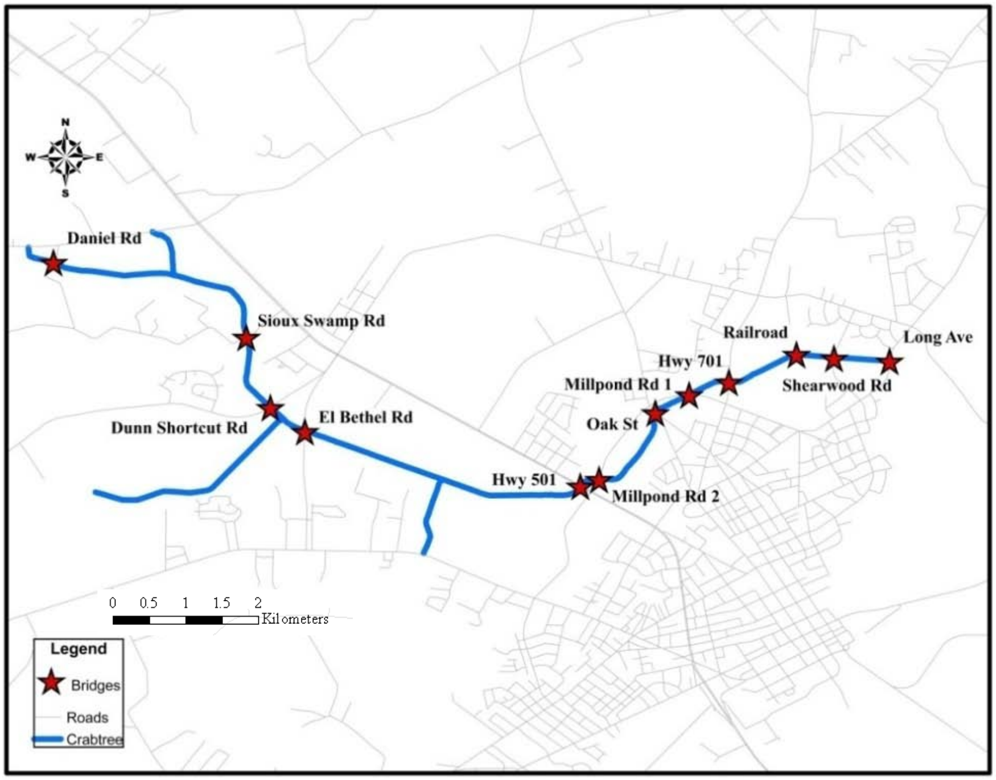


Figure 3.1. Map of bridges and culverts included in the model.

Available data sources

Mathematical computations involved in a hydraulic model can either be theoretically based or empirically based. Modeling in general has greatly improved in recent years with the advancement of geographic information systems (GIS), radar-based rainfall estimation using next generation radar (NEXRAD), high resolution digital elevation models (DEMs), distributed hydrologic and hydraulic models, and the online delivery systems by which information is made available (Knebl et al., 2005).

Different types of models and tools were compiled to form a collective, more comprehensive model. The compilation of tools for this project included ESRI's ArcMap™, USDA's WinTR-55, and HEC-RAS (ArcMap™ and WinTR-55 were used to derive input data for HEC-RAS).

For this study, hydrographs were obtained using Win TR-55 computer program. WinTR-55 models single rainfall and direct surface runoff events; it is available online from the NRCS (http://www.nrcs.usda.gov/). WinTR-55 uses the TR-20 (NRCS, 2002) model for all of the hydrograph procedures: generation, channel routing, storage routing, and addition; it does not model inputs from groundwater or ice. Crabtree is located in a coastal region of South Carolina where water tables are relatively high. Excluding contributions from groundwater may underestimate the actual discharge in the stream.

Land use data for this study was obtained from the National Land Cover Database (MRLC, 2009) dataset. NLCD is a land cover database produced by the Multi-Resolution Land Characteristics Consortium, an effort by several federal agencies to provide the nation with digital land cover and ancillary data (MRLC, 2009). Information pertaining to the soils in the watershed was obtained from Web Soil Survey (WSS) (http://websoilsurvey.nrcs.usda.gov). WSS provides access to the largest natural resource information system in the world (USDA, 2008). NRCS provides online data for more than 95% of the counties in the US. Table 3.1 contains the data sources utilized by this project as well as the data that was obtained from the respective source.

Hydrograph generation

Hydrographs were generated using WinTR-55, a single event rainfall-runoff small watershed hydrological model. The watershed was broken up into subareas and reaches. Subareas and reaches either drain to other reaches, or to the watershed outlet. For this model, the outlet is set at the Long Ave. Bridge at Crabtree Canal (downstream extent of Reach 1) (Figure 3.2).

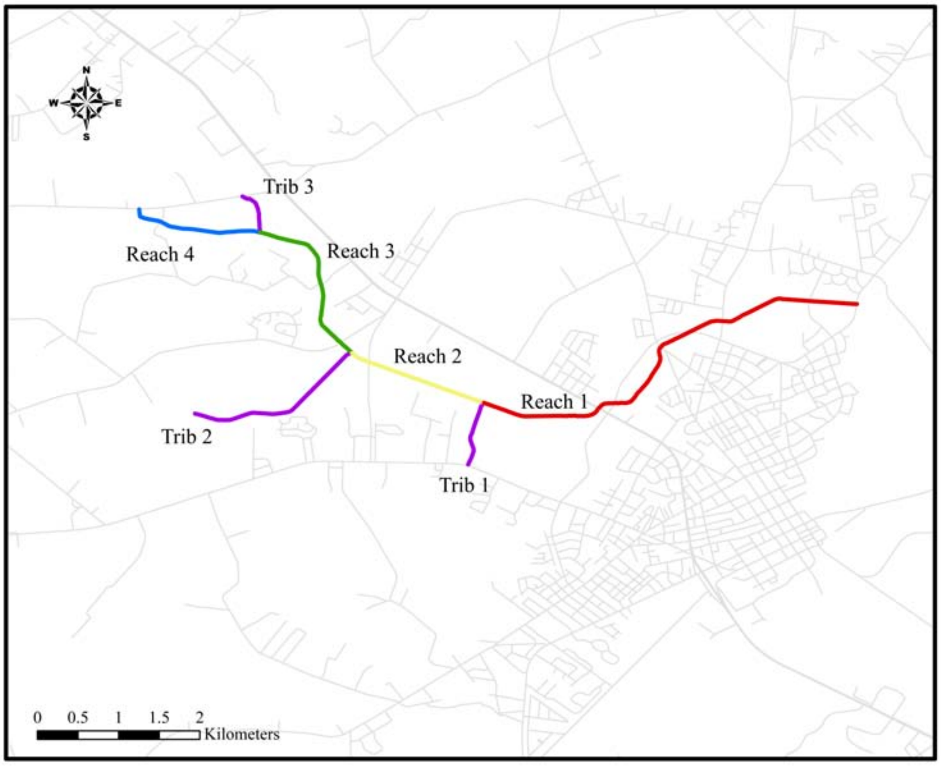


Figure 3.2. Schematic of reaches and tributaries that were modeled.

Flow hydrographs were needed at the top of Reach 4 and at the top of all the tributaries. Within WinTR-55, subareas were defined that drained into the tops of the tributaries and the top of Reach 4. Watershed areas were delineated within a GIS environment using ArcHydro (Maidment, 2003) and a digital elevation model (DEM) of the study region. The DEM was obtained in raster format from the Geospatial Data Gateway (http://datagateway.nrcs.usda.gov/GatewayHome.html), a website hosted and maintained by the United States Department of Agriculture (USDA). The DEM used in this study was of 30-meter spatial resolution and part of the National Elevation Dataset (NED) (USDA, 2006).

The National Land Cover Database (MRLC, 2009) was used as a source of land cover information for the study region. Land cover data were downloaded as digital raster files from the National Map Seamless Survey (http://seamless.usgs.gov/), a website hosted and maintained by the United States Geological Survey (USGS). The dataset comprises a raster dataset of 50-meter spatial resolution, each pixel representing a specific land use. Land cover in the Crabtree Canal watershed and those subwatersheds that contribute to flow in the four modeled tributaries were estimated using an overlay function of watershed extent and land cover data.

WinTR-55 produces both a peak flow and a time to peak for all subareas and reaches as well as hydrographs that can be exported into Microsoft Excel ® as tables or graphs. Flow hydrographs were used in the unsteady flow analysis. Different rainfall events can be chosen to produce various hydrographs. The unsteady flow data used to run simulations in this study corresponded to a storm with a return period between 2 and 5 years. A rainfall distribution type III was used (SCS, 1986). WinTR-55 was used to generate hydrographs for the top of each tributary and the top of Reach 6. See Appendix D for more detail on the hydrographs used. Hydrographs were generated using land use details, soil types and rainfall data for the study area. The chosen storm event was used because it ensured that at least 0.15m (0.5ft) of water always flowed over the floodplain for every floodplain configuration modeled. Unsteady flow simulations would go unstable if a dry condition on the floodplain was encountered.

Table 3.1. Data sources utilized and the respective data obtained.

Data Source	Data Obtained
National Land Cover Database	Land use data
Web Soil Survey	Soil information
Geospatial Data Gateway	DEM used to delineate watershed areas
NRCS	Horry Co. precipitation data

Channel modification

A two-stage channel design modification was applied to Reaches 1, 2, and 3 (See Figure 3.2). The tributaries that were included in the model were not modified. Floodplain ratios of 2, 3, 5, 7, 10 and 20 were modeled to give a range of results. At the proposed modification site (between Millpond Bridge 2 and Oak St Bridge), an incipient floodplain was starting to form. This floodplain was approximately 1.1 m (3.6 ft) above the channel bed. The channel discharge that would produce a depth of flow of 1.1 m (3.6 ft) at this point was found and was used to determine the depth of flow in the other reaches of the system. The depth of the main channel of the two-stage design was made to be the same depth that corresponded to the depth of flow that caused the incipient floodplain. A steady discharge of 4.6 m^3/s (162 cfs) at Long Ave Bridge is the flow that corresponded to a 1.1 m (3.6 ft) flow depth at the proposed modification site. This discharge corresponded to a storm much smaller than a 2 year storm. WinTR-55 was used to determine what proportion of the total flow (4.6 m^3/s, flow at Long Ave. Bridge) was contributed by each of the tributaries. Crabtree is an excavated channel that is periodically dredged; it is of earthen base, straight and uniform; hence a Manning's n value of 0.022 (based on Chow, 1959) was used for the main channel. Depending on if and what of kind vegetation is planted on the floodplain after reconfiguration, the Manning's n value would change on the floodplain. After modification, native plants and trees would be planted on the floodplain so a light brush with trees condition would be present on the floodplain. Chow (1959) suggested a minimum Manning's n of 0.04 for a light brush with trees condition. The minimum value was chosen because immediately after reconfiguration the vegetation wouldn't be very thick and would inflict the minimum amount of friction on overbank flow. The side slopes of the main channel would not be disturbed below the top of the main channel. Excavation would only be done to alter the floodplain. A 2:1 side slope was used from the floodplain elevation to the original channel. According to USDA-NRCS (2007) the angle of repose for a Meggett loam (dominate soil type) is 32.5° therefore a 2:1 side slope would be acceptable. The original channel bed elevation was not changed. Unsteady flow simulations were performed for the following floodplain configurations:

a) The existing geometry,

b) Floodplain ratio of 2,

c) Floodplain ratio of 3

d) Floodplain ratio of 5

e) Floodplain ratio of 7

f) Floodplain ratio of 10

g) Floodplain ratio of 20

Mean velocities (Equation 3), average depth of flow, the average shear stress (Equation 2) exerted on the channel boundaries and a weighted average shear stress (total shear stress) (Equation 4) across the width of the channel at each cross section were chosen to quantitatively compare the different simulated floodplain configurations. Over the course of the unsteady flow simulation, the mean velocity, hydraulic depth and shear stress was noted at the point in time when the water surface elevation was the maximum for each cross section. An average of the values at each cross section was calculated for each modified reach. The minimum and the maximum values for mean velocity, hydraulic depth and shear stress were also recorded for each modified reach.

Total shear stress across the width of the channel is calculated by:

$$\bar{\tau} = \frac{\sum_{i=1}^{n} l_i \tau_i}{\sum_{i=1}^{n} l_i} \qquad (4)$$

Where: $\bar{\tau}$ = total shear stress (N/m²) and

τ_i = shear stress per unit cross sectional width (N/m^2)

l_i=Unit cross sectional width (m).

If the length of the left floodplain was 5 m, the length of the right floodplain was 6 m, the length of the main channel is 3 m, the average shear stress on each floodplain was 1 N/m² and the average shear stress in the main channel was 2 N/m², then the total shear stress would be calculated as follows:

$$\bar{\tau} = \frac{\sum_{i=1}^{n} l_i \tau_i}{\sum_{i=1}^{n} l_i}$$

$$\bar{\tau} = \frac{5m*1\frac{N}{m^2} + 6m*1\frac{N}{m^2} + 3m*2\frac{N}{m^2}}{5m+6m+3m}$$

$$\bar{\tau} = \frac{5\frac{N}{m} + 6\frac{N}{m} + 6\frac{N}{m}}{14m}$$

$$\bar{\tau} = 1.2\frac{N}{m^2}$$

While stream bed elevation is determined by the balance between sediment supply and the sediment transport capacity, channel stability requires that the shear stress exerted by discharge remain below the critical shear stress of the channel bed (Clark and Wynn, 2007). A critical shear stress was calculated for the channel bed by using a method outlined in NRCS’s Stream Restoration Design National Engineering Handbook (USDA-NRCS, 2007). Critical shear stress for the channel was determined to be 7.2 N/m² (1.5 lb/ft²). Critical shear stress is compared to shear stress predicted by HEC RAS in order to determine where potential zones of instability are in the system. Steps for determining critical shear stress are given in Appendix E.

Increase in storage volume among the different floodplain ratios was also determined. There was no way to determine what the existing storage volume is in the system; however, in HEC RAS, one could determine the volume of soil excavated when modifications are done to the existing channel. The increase in storage volume was taken to be equal to the volume of soil excavated.

Cost-benefit analysis

In order to quantify the cost efficiency of the modifications preformed on the channel, a cost to benefit ratio index was derived and used to compare different modification options. Six different modification scenarios were modeled. For each scenario, FPRs of 2, 3, 5, 7, 10, and 20 were modeled. Scenario 1 included only Reach 1 being modified. Scenario 2 included Reaches 1 and 2 being modified. Scenario 3 included Reaches 1, 2, and 3 being modified. Scenario 4 included all of the tributaries and Reach 4 being modified. Scenario 5 included all of the tributaries, Reach 3 and Reach 4 being modeled. Scenario 6 was if all of the tributaries and all of the reaches being modeled were modified. Figures 3.3, 3.4, 3.5, 3.6, 3.7, and 3.8 are maps of all scenarios. In all the maps, the red line represents the reach that was modified while the black line represents the reaches that were left untouched.

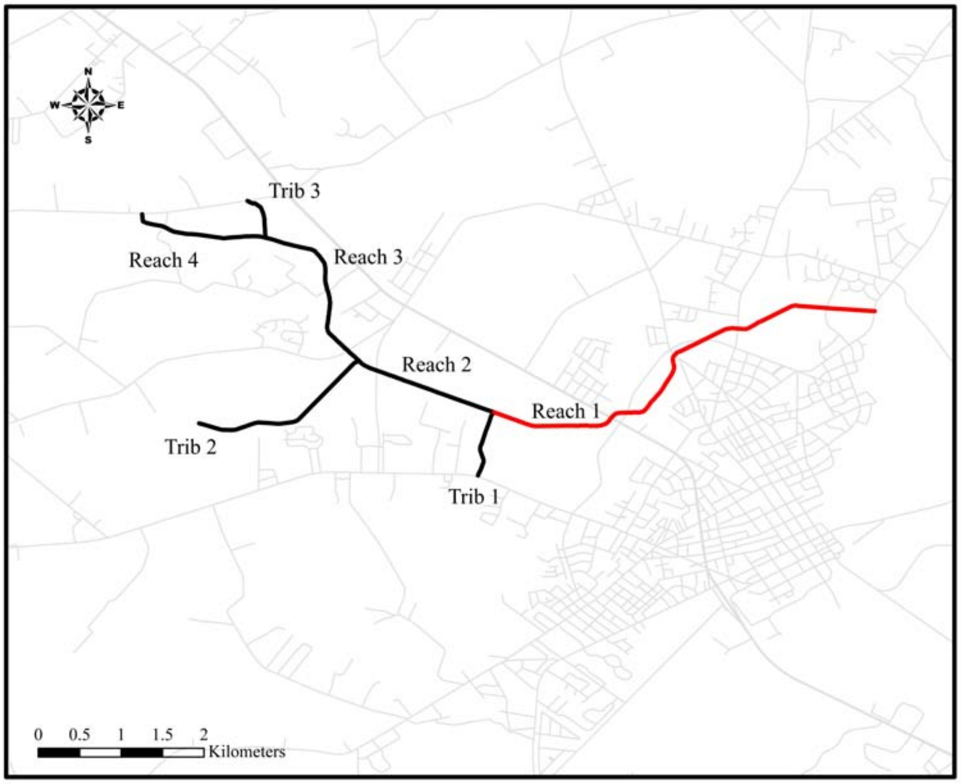


Figure 3.3. The first scenario involved modifying Reach 1.

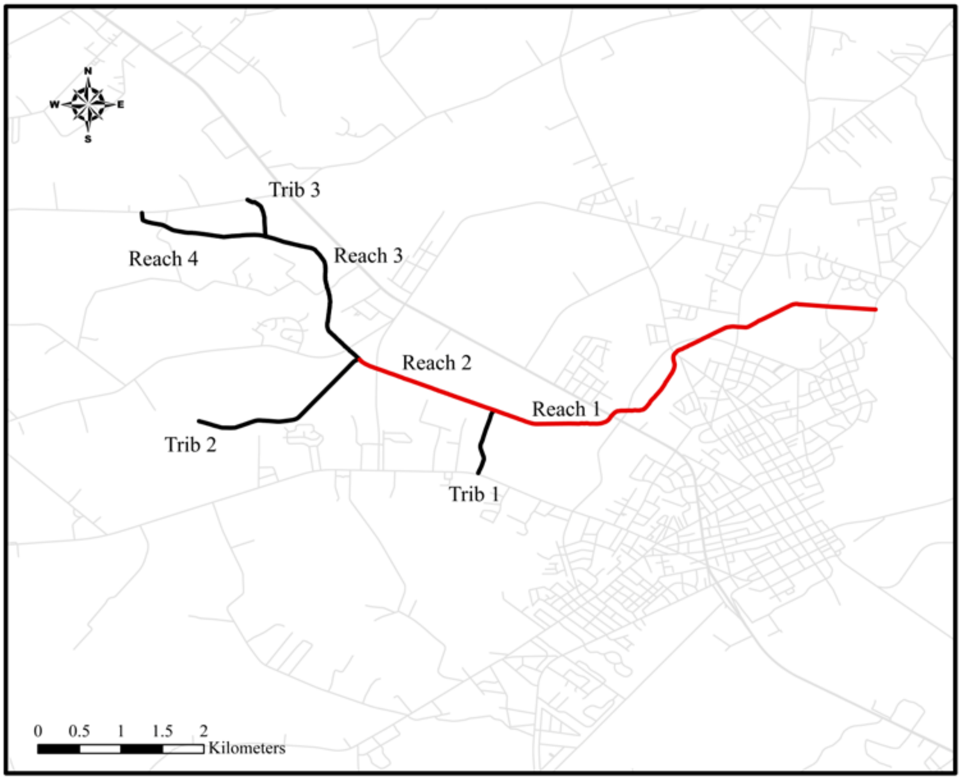


Figure 3.4. The second scenario involved modifying Reaches 1 and 2.

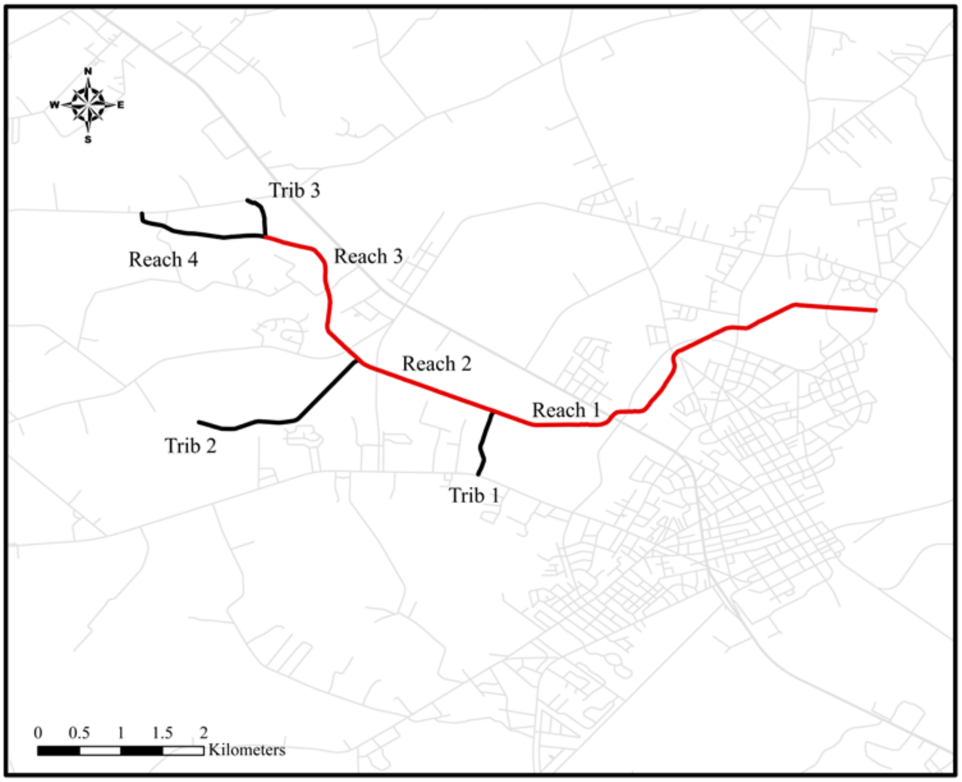


Figure 3.5. The third scenario involved modifying Reaches 1, 2, and 3.

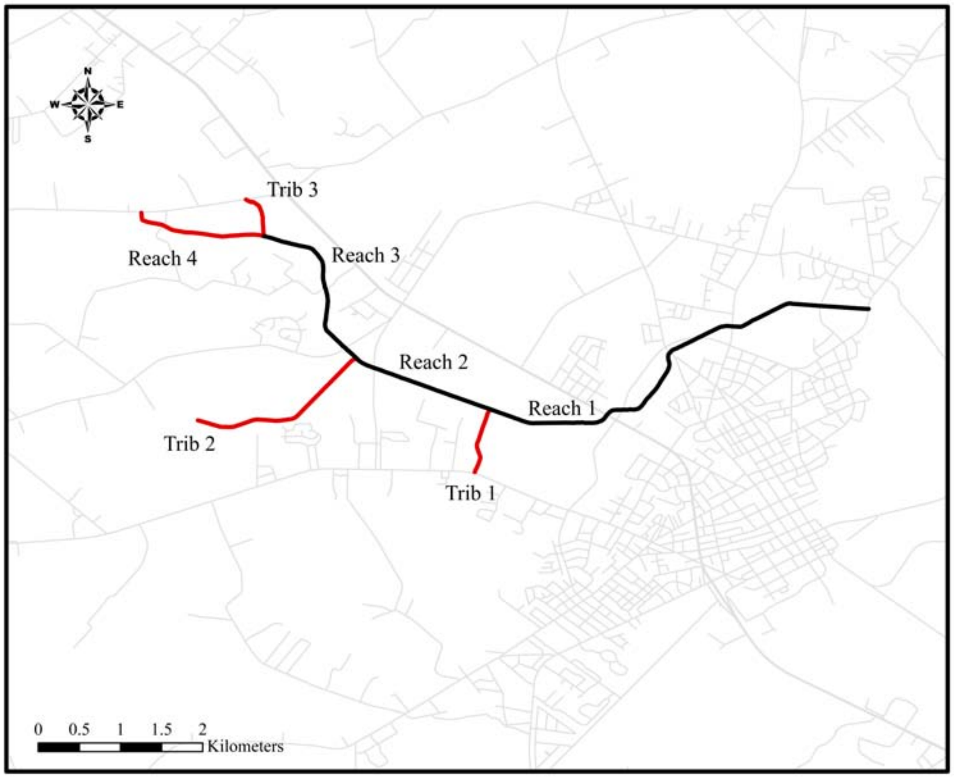


Figure 3.6. The fourth scenario involved modifying the tributaries and Reach 4.

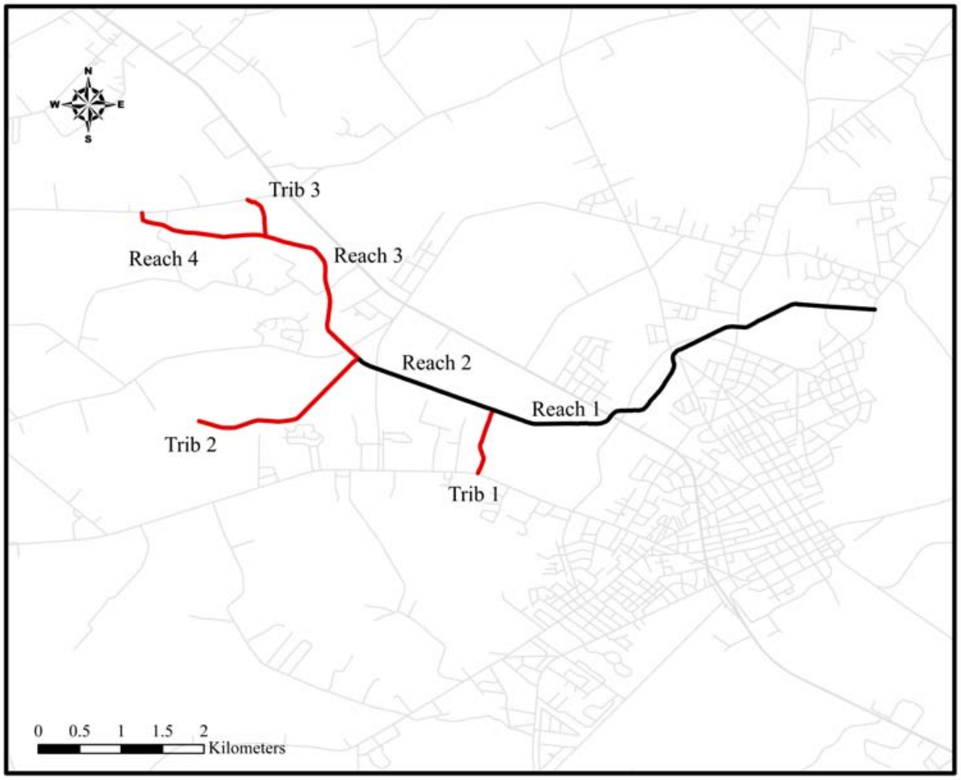


Figure 3.7. The fifth scenario involved modifying the tributaries, Reach 4 and Reach 3.

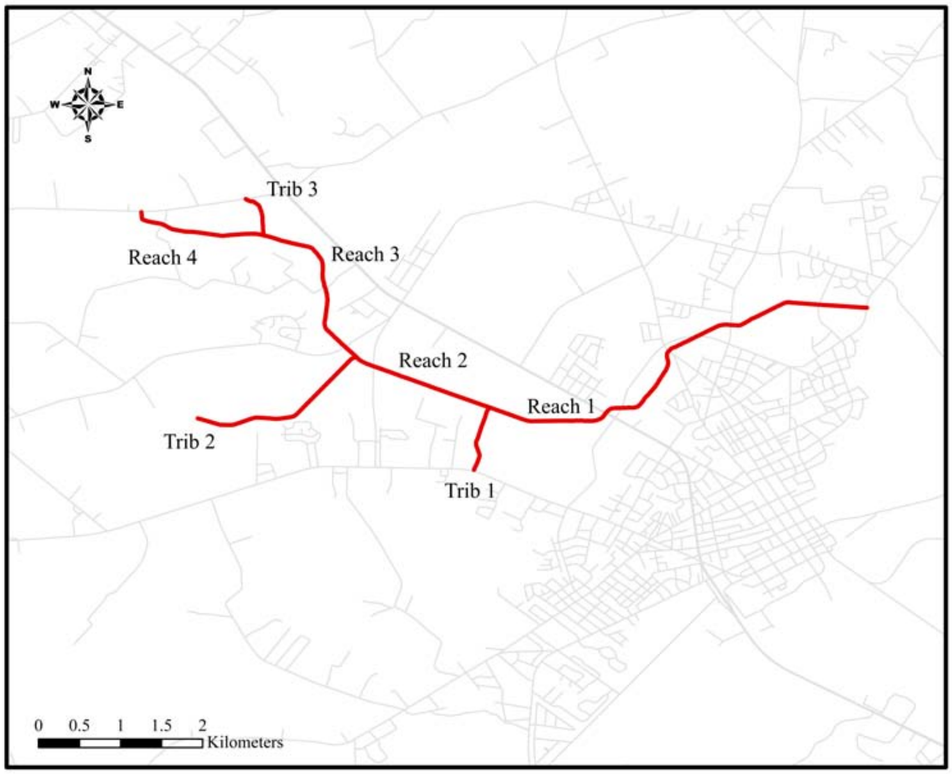


Figure 3.8. The sixth scenario involved modifying the entire system.

The volume of soil excavated was chosen as the variable that quantified the cost of providing additional floodplain. The reduction in main channel shear stress from the current geometry was chosen as the variable to quantify the benefit of increased floodplain. The ratio was calculated as cost over the benefit, with units $\frac{m^3}{N/m^2}$. The units themselves are meaningless and do not have any effect on the results of the cost-benefit analysis as the cost-benefit ratio is just an index for the comparison of various scenarios. However, they are consistent for each scenario that was compared and presented here for completeness.

CHAPTER FOUR

RESULTS

The mean velocity, the hydraulic depth, the shear stress and the total shear stress all decreased as the width of the floodplain increased relative to the top width of the main channel (Figures 4.2, 4.5, 4.6 and 4.7). The change in mean velocity, hydraulic depth, shear stress and total shear stress was greater between the smaller floodplain ratios such as 2, 3, and 5. There was a smaller change in the above-mentioned variables for larger floodplain ratios. There was a 13% decrease in shear stresses imposed upon the main channel between the trapezoidal channel and FPR 2 configuration. The greatest decrease in main channel shear stress occurred between FPR 3 and FPR 5 scenarios and was approximately14%. The greatest decrease in total shear stress between the trapezoidal shape and FPR 20 configuration occurred in Reach 1 and was an 86% decrease. Values used to determine main channel average shear stress values presented in Figure 4.6 are biased by extremely high shear stress values at specific zones in the stream network. These high shear stress values occurred at points of inflection in bed profile or where the bed transitioned to a steeper slope (Figure 4.1). The high shear stress values were only encountered in the main channel. Shear stresses on the floodplain decreased with an increase in FPR values (dashed lines in Figure 4.6). The total shear stresses over the entire channel including the floodplain were not biased by high shear stress values caused by inflection points in the stream bed profile. This is due to the fact that the floodplain is relatively wide compared to the main channel therefore a weighted average of shear stresses over the entire length of the channel cross section is dominated by floodplain shear stresses (Figure 4.4).

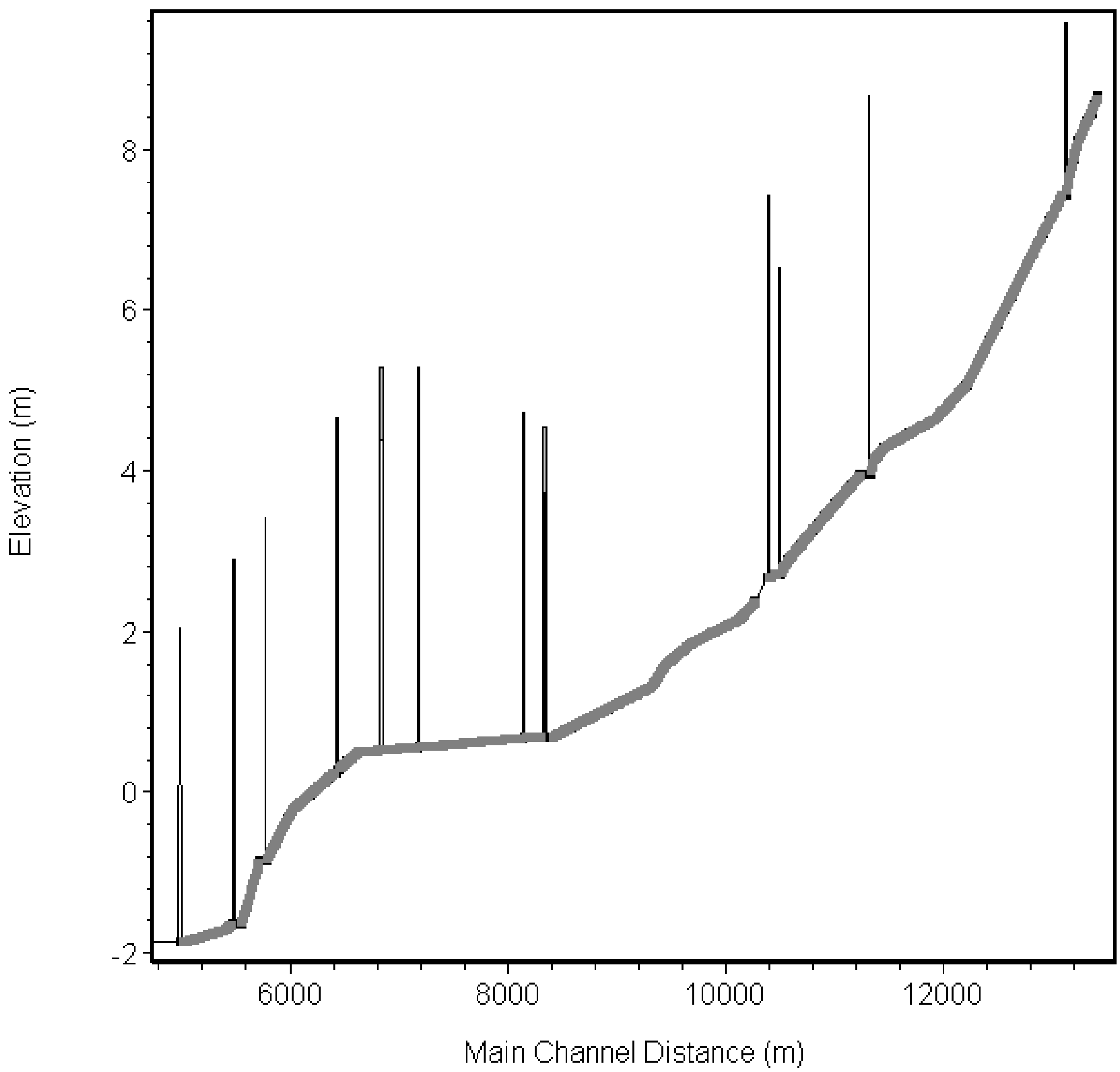


Figure 4.1. Streambed profile showing points of inflection and points where slope transitions to a steeper slope. Vertical lines represent bridges.

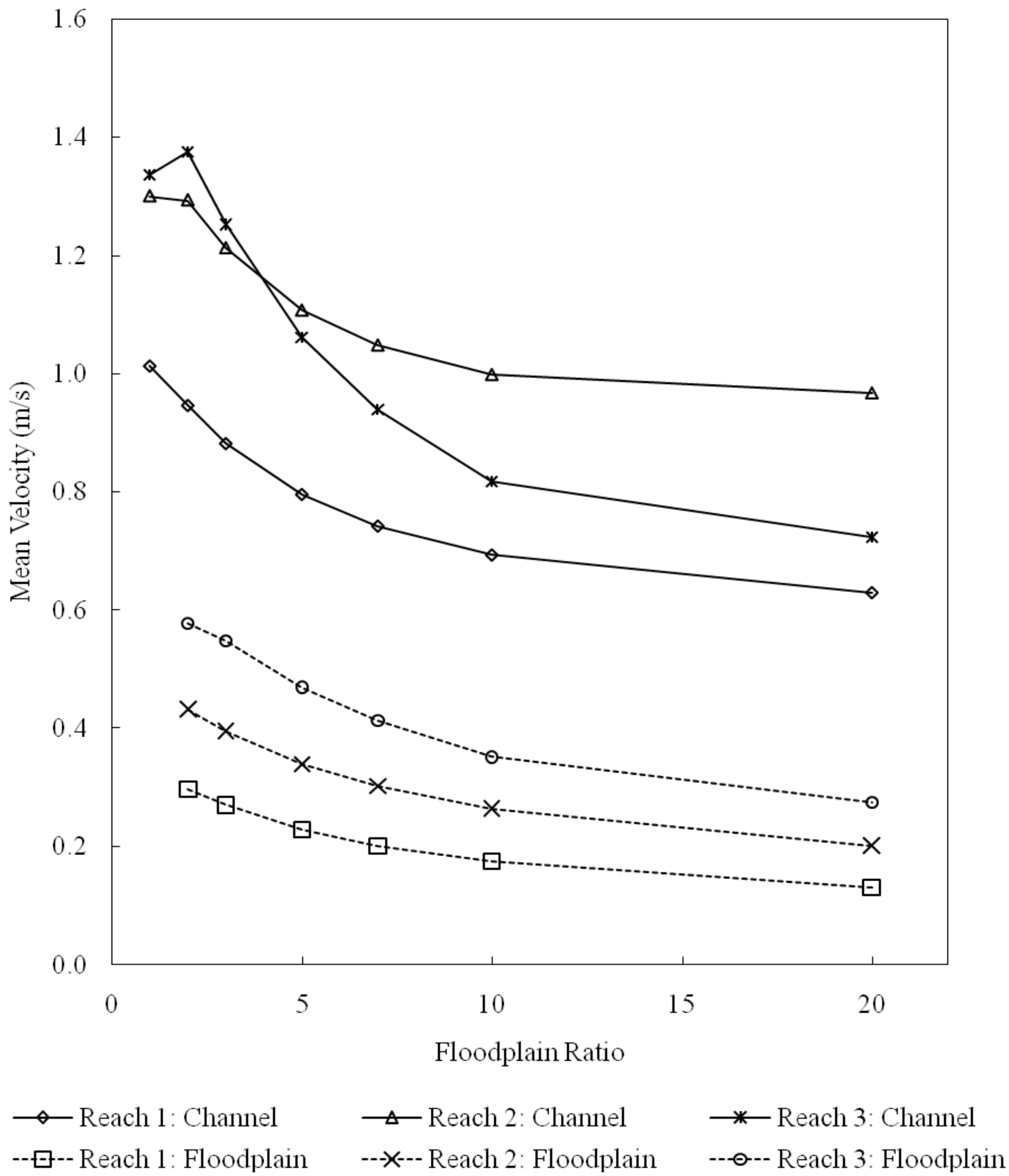


Figure 4.2. Mean velocity versus floodplain ratio for a simulated 2-year storm event. Solid lines represent the main channel and dashed lines represent floodplains.

There was a slight increase in mean velocity that occurs in the main channel of Reach 3, between current geometry and a FPR of 2. This increase is due to an increase in hydraulic radius between the current geometry and the modified FPR2 conditions. HEC RAS separates the calculations for the main channel from calculations for the floodplain. The designated bank stations for the existing geometry provided a larger wetted perimeter which led to a smaller hydraulic radius than the FPR2 geometry. (Figures 4.3 and 4.4)Recall Equation 3:

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad \text{Equation 3}$$

It follows from Equation 3 that if the hydraulic radius of the system increases that the velocity will also increase. The same phenomenon is observable for shear stress results as shown in Figure 4.3. Recall Equation 2 for average shear stress:

$$\tau = \gamma R S \qquad \text{Equation 2}$$

The increase in shear stress in the main channel of Reach 3 between current and FPR2 data is also due to an increase in hydraulic radius.

Critical shear stress was also plotted on the graphs containing shear stress and total shear stress. In Figure 4.6, the main channel of Reach 1 and all floodplain areas are below the critical shear stress value. Lines representing the main channels of Reaches 2 and 3 intersect the critical shear stress line between FPR 3 and FPR 5 data points.

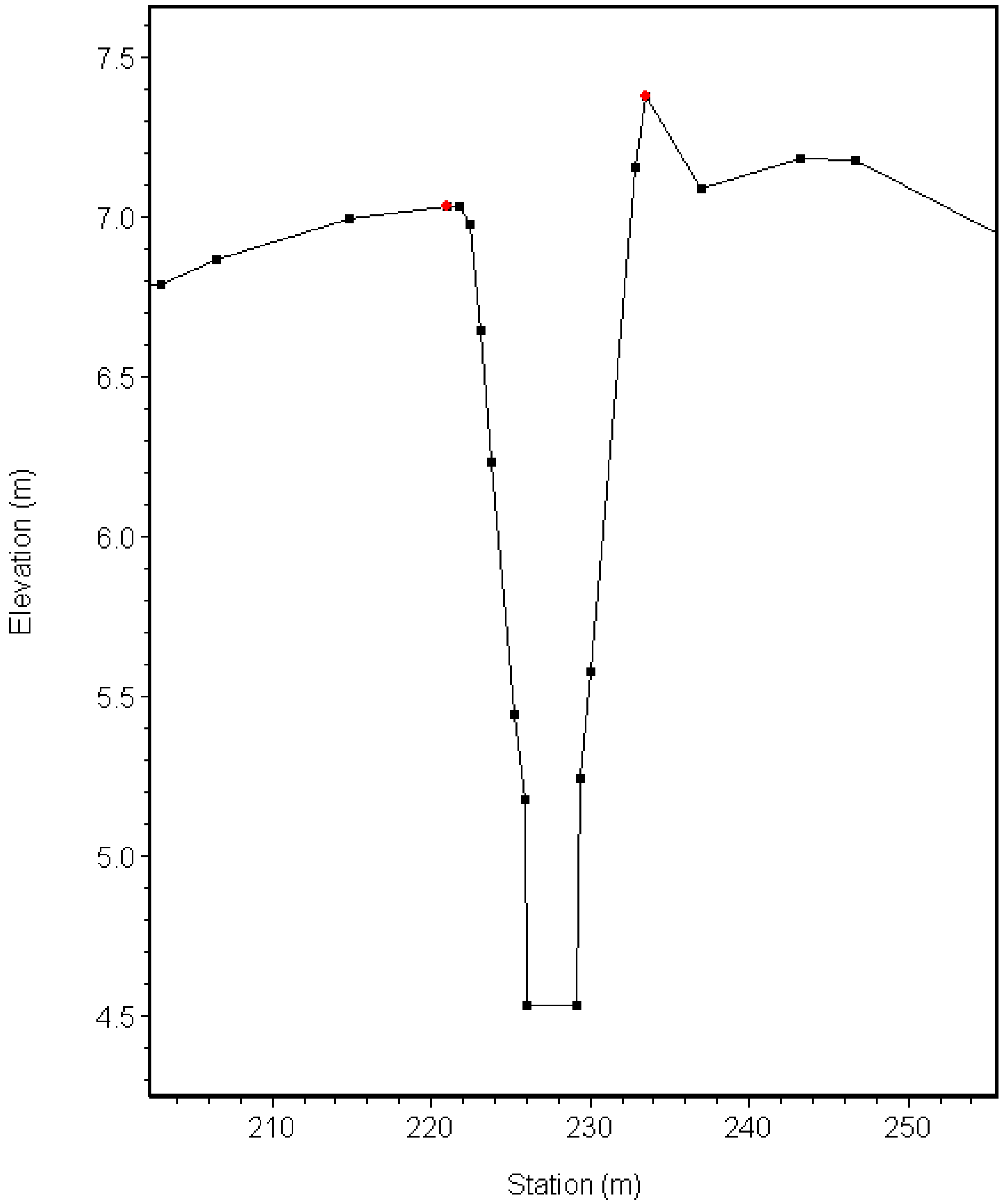


Figure 4.3. HEC RAS defines the main channal as the length of cross section between designated bank stations (red dots). Bank stations for existing geometry provide for a main channel with a larger wetting perimeter.

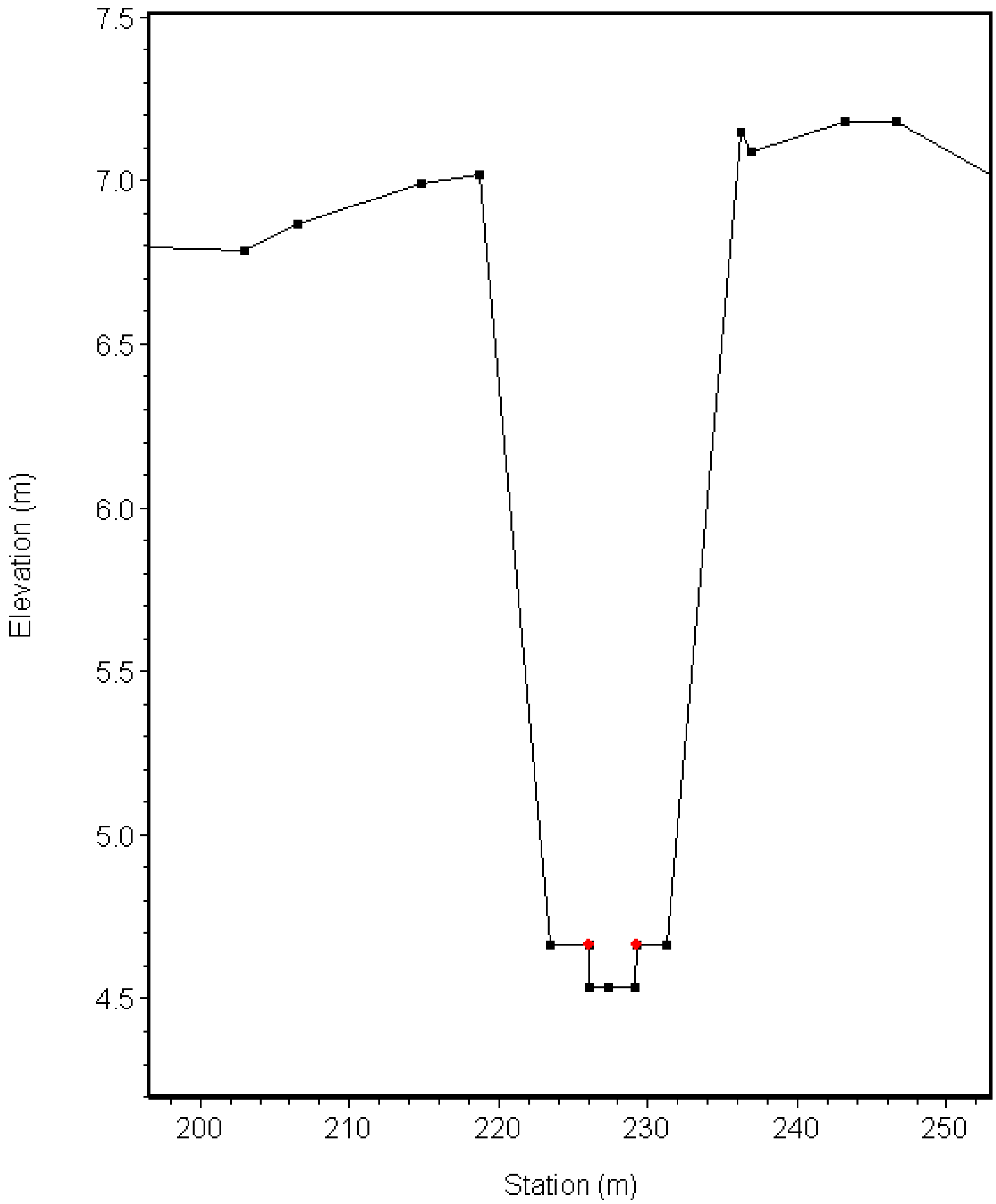


Figure 4.4. Bank stations (red dots) for FPR 2 geometry provide for a much smaller main channel wetted perimeter.

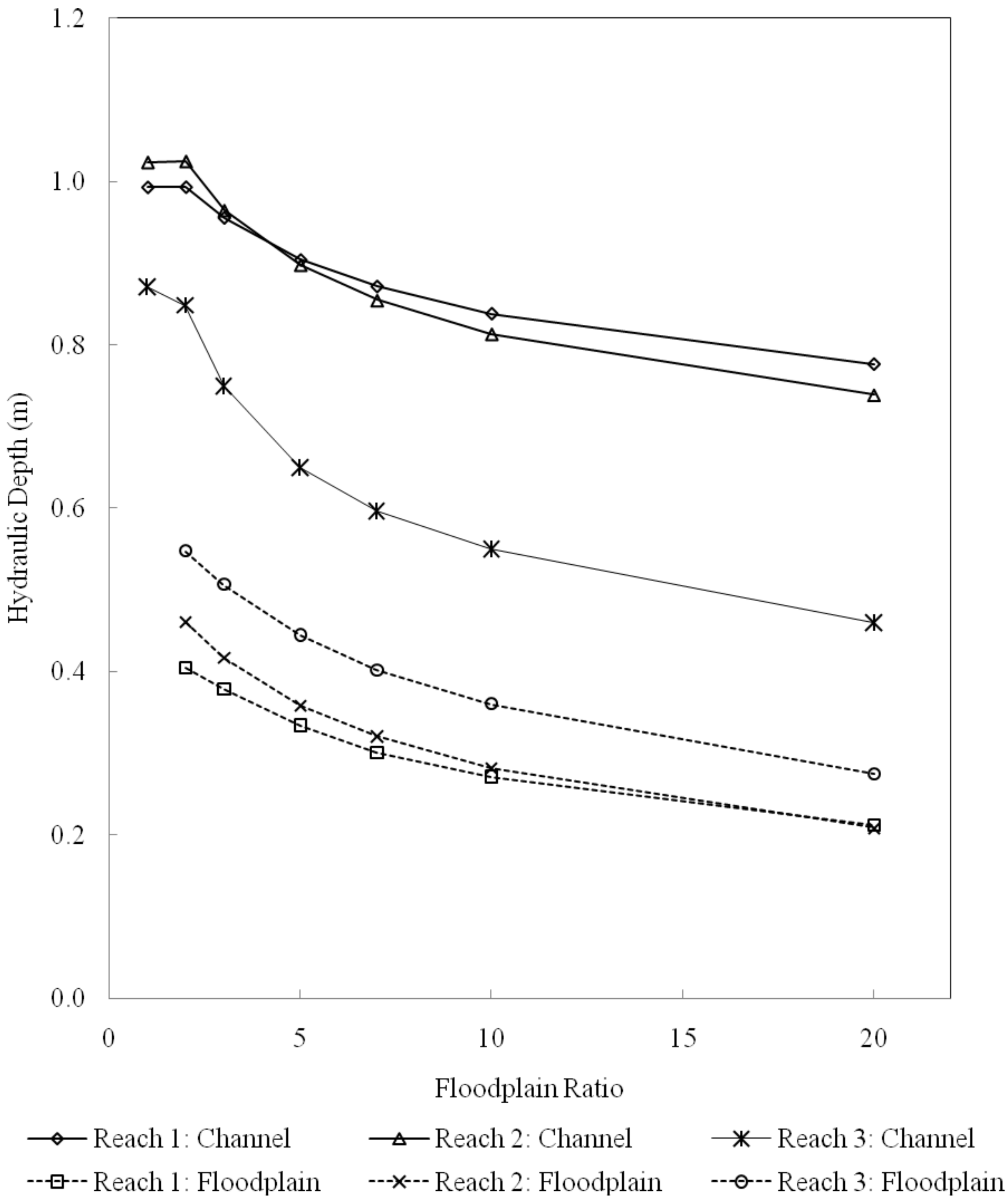


Figure 4.5. Hydraulic depth versus floodplain ratio for a simulated 2-year storm event.

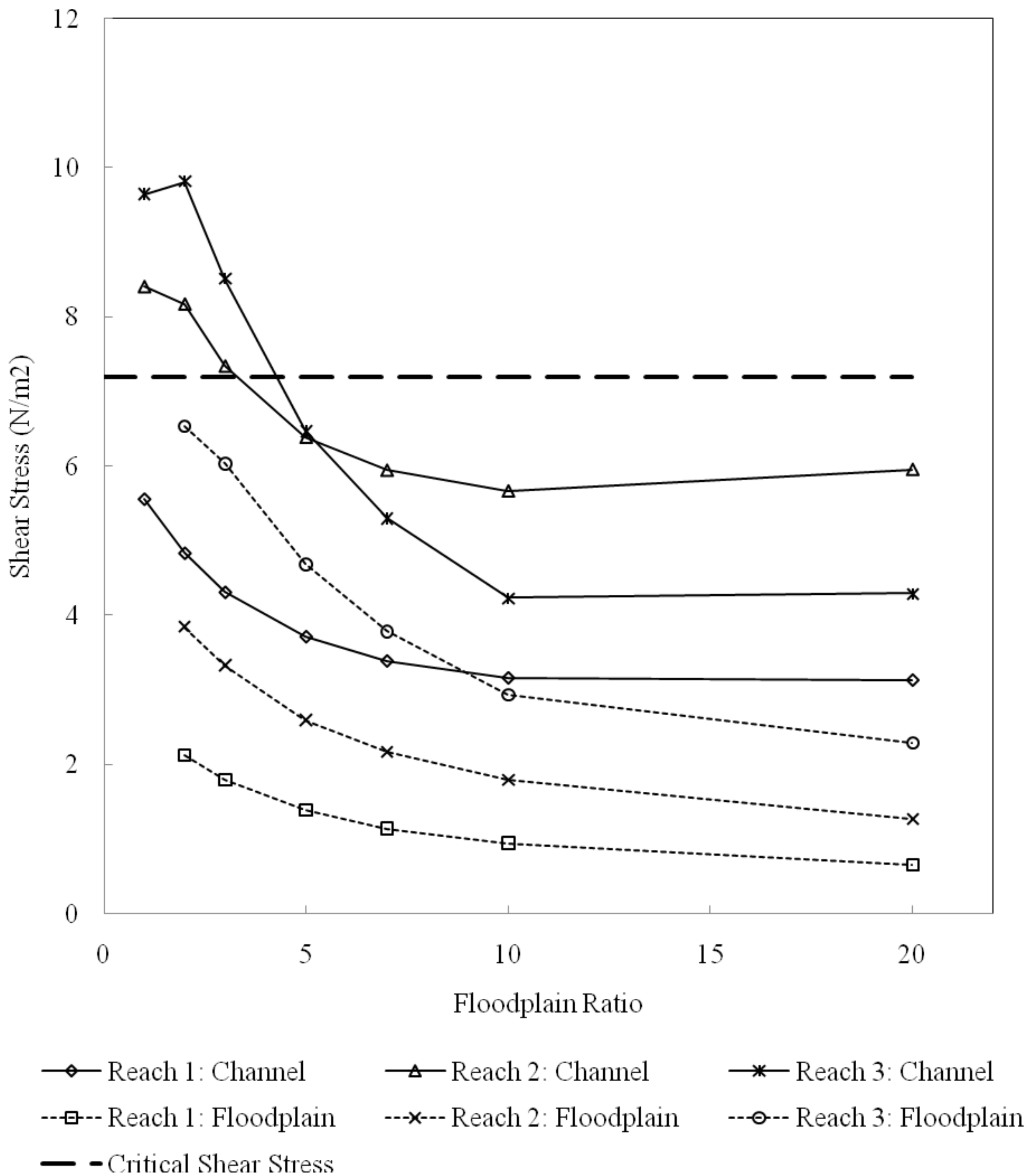


Figure 4.6. Shear stress versus floodplain ratio for a simulated 2-year storm event.

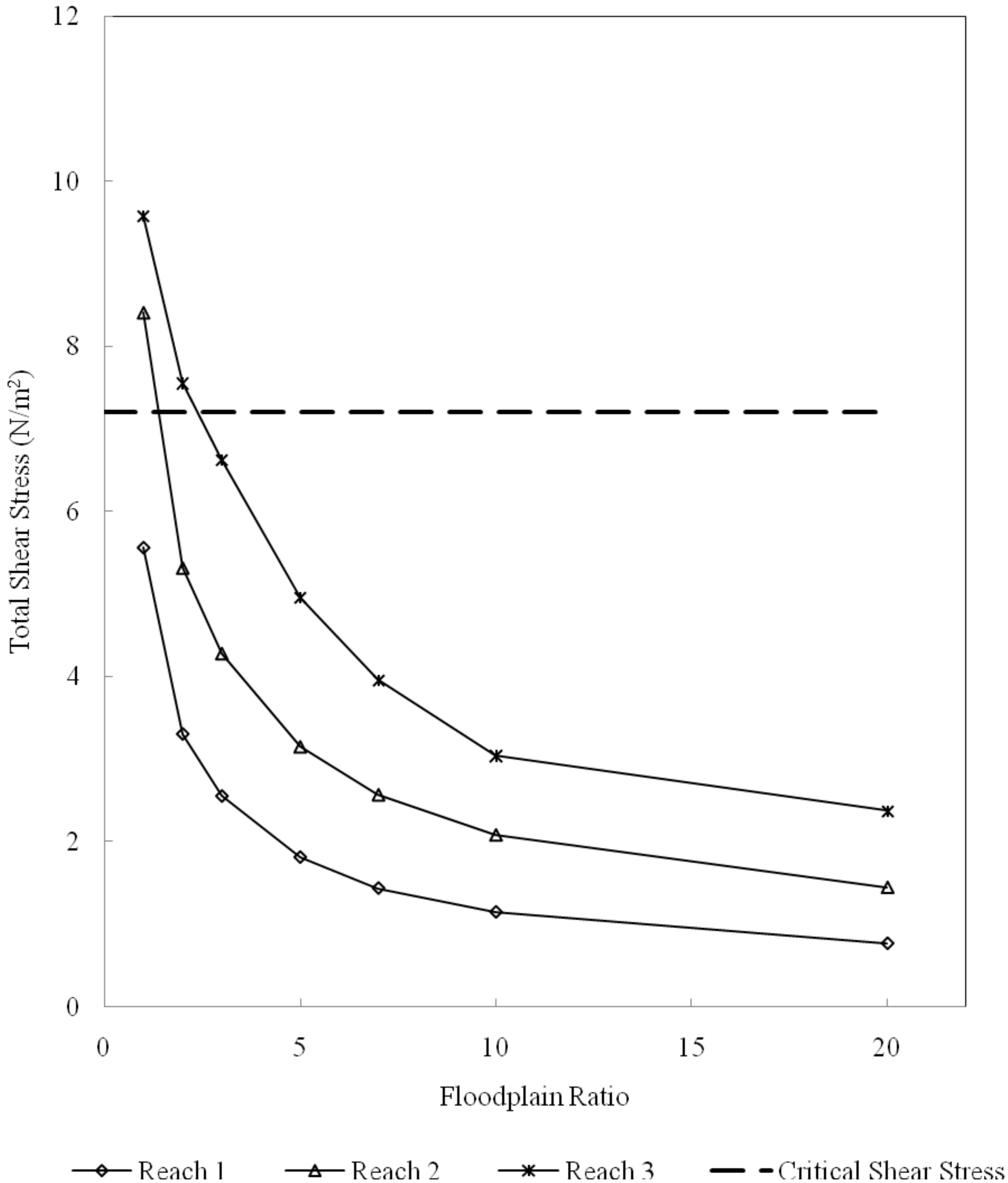


Figure 4.7. Total shear stress (channel and floodplain) versus floodplain ratio for a 2-year storm event.

Along with a general decrease in mean flow velocity, maximum flow depth and shear stress in the channel, an additional benefit to altering the floodplain configuration is additional storage volume. An FPR of 2 increases the flood storage up to 230,000 m^3 (300,000 yd^3). An FPR of 3 would produce a 79% increase in storage volume over a FPR 2 configuration. A FPR 5 configuration would produce a 220% increase; a FPR 7 would produce a 360% increase; a FPR 10 would produce a 563% increase and a FPR 20 would produce a 1420% increase in storage volume within the stream channel. Depending on the potential for flooding in the area at hand, one should consider a larger floodplain to accommodate larger volumes of flood waters.

The cost to benefit ratio previously described was used to determine which scenario would produce the smallest cost per unit of benefit. A matrix (Table 4.1) was constructed that contained the scenario on the vertical axis and the FPR on the horizontal axis. The cost to benefit ratio was entered into the matrix cells and the smallest ratio was found. Lowest cost-benefit ratio is seen for FPR2 with all tributaries modified.

Table 4.1. Matrix containing the cost to benefit ratios for a cost-benefit analysis.

	Flood Plain Ratio					
Scenario	FPR2	FPR3	FPR5	FPR7	FPR10	FPR20
Reach 1	1649	2391	3763	5059	7288	16924
Reaches 1 & 2	1951	2886	4268	5524	7461	16087
Reaches 1, 2, &3	2382	2967	5015	4287	4724	11232
Tributaries	**209**	238	231	310	411	749
Tributaries & Reach 3	318	359	355	462	606	1110
Tributaries, Reaches 1, 2, 3	1085	1218	1508	1881	2217	5118

The lowest cost-benefit ratio corresponded to the scenario where only the tributaries were modified to a FPR of 2. The highest cost-benefit ratio corresponded to the scenario where only Reach 1 was modified to a FPR of 20 For all of the FPRs, the scenario where only the tributaries were modified had a 93% smaller cost-benefit ratio than when all the main stem reaches were modified.

Despite the fact that the scenario where only the tributaries were modified had the smallest cost to benefit ratio, this scenario did not significantly reduce the main channel shear stress in Reaches 1, 2 or 3 (Figure 4.8). There was no change in the main channel shear stress in Reaches 1, 2 and 3 when only the tributaries were modified to an FPR of 2.

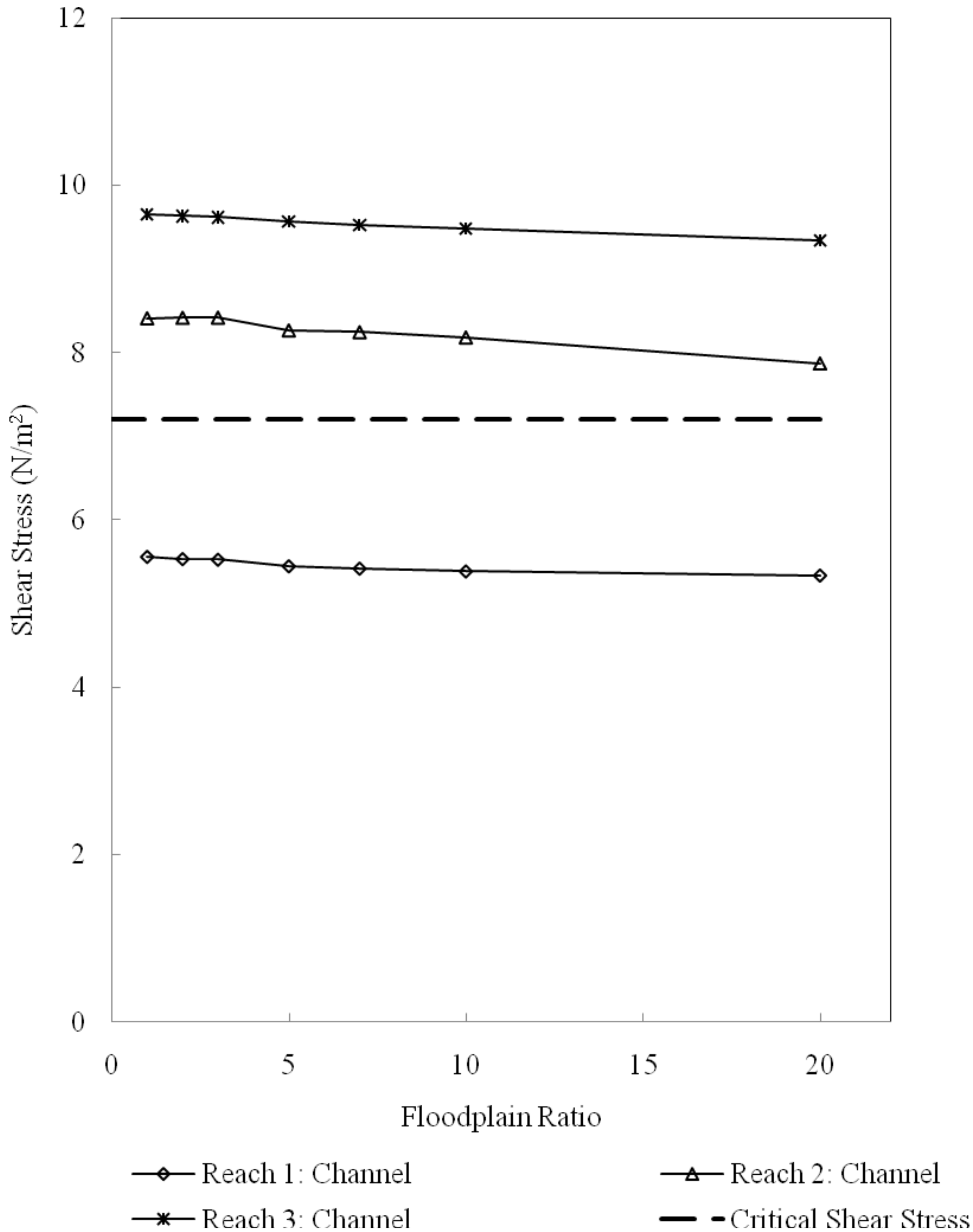


Figure 4.8. Main channel shear stress corresponding to the scenario when only tributaries were modified.

CHAPTER FIVE

DISCUSSION & CONCLUSIONS

The general trend is for the mean velocity, the hydraulic depth and the shear stress exerted to all decrease in the main channel as well as in the floodplain as the floodplain ratio increases. Floodplain ratios of 2, 3, 5, 7, 10 and 20 were compared with the current channel morphology. The results of this modeling effort showed that as floodplain width increased, mean channel velocity, hydraulic depth and shear stress decreased. Once the FPR reached 10, there was no significant decrease in the mean velocity or the shear stress. Only one size storm event was simulated in this study. There were stability issues when larger storms were simulated. One of the objectives of this study was to quantify the relative performance of different floodplain configuration. Since every floodplain configuration was modeled using the same size storm, the performances of the different floodplains could be compared relative to each other. Although the degree of improvement was not consistent with the increase in floodplain ratios, there was always an improvement to the system attributes when the floodplain ratio was increased, a result consistent with Ward et al. (2008).

Points of inflection and points where the bed profile transitioned to a steeper slope caused high values of shear stress to occur in the main channel due to the flow approaching critical depth; this resulted in average shear stress values in the channel that were slightly biased by a few larger values. From field observations incurred during manual surveys of Crabtree Canal, we believe that super critical flow does not occur in the regions suggested by model output. The default solution methodology that HEC RAS uses for unsteady flow simulations is for subcritical flow. HEC RAS has an option to model super and subcritical flow, but HEC (2002) suggests only using this option if it is known with a high degree of certainty that the flow goes from subcritical to supercritical flow. The high values of shear stress were generated when the solution approaches supercritical flow. The zones of high shear stress could possibly be zones of instability in the channel and would require more attention should stream restoration efforts take place. The input geometry of the bed could have been altered to exclude all points of inflection but alterations of this extent would have changed the modeled geometry to be significantly different from the actual stream geometry. Further research should be conducted to determine how removing points of inflection and further smoothing changing in bed slope would change modeling results and if stream flow is actually approaching critical depth at points of inflection and points where bed slope drastically increases.

Critical shear stress for Crabtree Canal was determined to be 7.2 N/m^2 (1.5 lb/ft^2). This value was greater than the predicted shear stress values for the main channel at Reach 1 and all floodplain areas. Reaches 2 and 3 were above the critical shear stress threshold. Once an FPR of 5 was reached, the shear stress values for these two areas dropped below the critical shear stress threshold. In-channel erosion could be occurring in these two upstream reaches, causing increased deposition in downstream extents of Crabtree Canal. Downstream sections of the Canal system that are influenced by tidal and backwater effects will result in lowered sediment transport potential in these zones due to slack water and reversing flows. . This could provide for better settling conditions for suspended sediments that originated in Reaches 2 and 3.

Results of the cost-benefit analysis suggest that a more economically efficient design would involve focusing restoration efforts in the headwaters of the system and not in the downstream extents of the system. Although Ahn et al. (2006) had different management goals; they also reported that naturalization efforts were more successful in the headwaters of the system. Ahn et al. (2006) reported that downstream water structures have a negative effect on aquatic plants that try to take root. Depending on the goal of restoration, a FPR greater than 2 may be desired, but scenarios involving modifying only the tributaries or the tributaries and some combination of the tributary reaches were more economically efficient than modification scenarios that comprised only modifying the main stem reaches of the system. There were other cost factors that were not considered in the cost-benefit analysis, including cost of earthwork, costs associated with hauling earthwork away from the job site, the market value of the land to perform the modifications, and labor costs. These additional costs may not have been constant for each different scenario used; however, the cost per unit of soil excavated was considered to be constant no matter what the volume of soil excavated. There were also other benefits to the modifications besides the decrease in shear stress across the channel. There was an aesthetic benefit to the local community as well as an ecological benefit to the surrounding ecosystem. It was difficult to quantify these types of benefits and therefore they were not included in the cost-benefit analysis. Although modification in the headwaters may cost less, there was not a significant decrease in main channel shear stress in Reaches 1, 2 and 3 when only the tributaries were modified. Existing conditions in Reaches 2 and 3 are above the critical shear stress threshold. Modifying only in the tributaries would not bring shear stress levels in Reaches 2 and 3 below erosive limits.

This study did not consider ecological components of channel alteration, or the provision of additional floodplain to a once incised drainage canal. Further work is needed to quantify the beneficial ecological impacts induced by channel modification on riparian flora and fauna such as improved nutrient recycling and increased habitat (Feyrer, 2006 and Ahn et al., 2006). Another benefit of additional floodplain that should be further researched is the provision of an aesthetic counter balance to urbanization (Searns, 1995).

In summary this study used a one-dimensional hydrologic model to compare the effects of different floodplain configurations on certain hydrodynamic parameters that relate to the fluvial functioning of the stream system. The model developed in this study was used to determine hydrodynamic conditions on the watershed driven by a hypothetical storm event and alternative channel geometry configurations. Relative performances of different floodplain configurations were quantified using the developed model and possible zones of instability were identified within Crabtree Canal. The economic efficiency the different floodplain configurations was also considered and it was determined that the most economically efficient configuration may not be the best scenario to choose due to its lacking in channel improvement over the entire system.

APPENDICES

Appendix A

Development of HEC RAS Model

Within a GIS environment, LiDAR data was used to develop a digital terrain model (DTM) both as a Trianguated Irregular Network (TIN) and as a raster (GRID) format. The DTM was applied to the watershed and the surrounding area. A stream centerline layer was then added using HEC-GeoRAS. Centerlines are used to assign river stations to cross sections and to display as a schematic in the HEC-RAS geometric editor. The centerline was broken up by reaches and tributaries in the drainage network. Centerlines were drawn from upstream to downstream; junction points connected each tributary to the main stem. After establishing the locations of the centerlines, each reach and tributary was assigned a unique reach and river name.

A flow path layer was created to identify the hydraulic flow path in the main channel, the left overbank, and the right overbank. The stream centerline was copied and used as the center flow path. The mirror feature tool was used to offset a copy of the center flow path on each side for the left and the right overbanks. The LineType tool was then used to identify each flow line as a left, channel, or right flow path. Like the stream centerlines, the flow path lines did not intersect.

The third layer added to the map was a layer showing the cross-sectional cut lines to show the location, position and the extent of the chosen cross sections. Cross sections were used to extract elevations along the channel. The cut lines were constructed from left overbank to right overbank and were drawn perpendicular to the direction of flow. Care was taken to ensure that cut lines did not intersect each other and that cross sections covered the entire floodplain. The centerline and flow paths were then exported from ArcMap™ into the geometry editor in HEC RAS. Since a GRID was used for the DTM, there were data points every 0.62 m (2 ft) apart on the cross sections. This produced more data points than were necessary for the model. Within the cross section editor of the geometry window, some of the data points were manually removed to decrease number of data points representing cross sections Figure A.1. is a 3D view of the entire model in HEC RAS.

In HEC RAS, the Bridge/Culvert editor window is accessed via the geometric data window. The location of the bridge on a given reach is input into the model by defining a river station for the location of the bridge. Required data for a new bridge include location of the bridge (river, reach, river station identifiers), short description of the bridge, bridge deck dimensions, dimensions of any bridge abutments (if they exist), pier dimensions (if they exist), and bridge modeling approach information. Optional information pertaining to the bridge such as debris blockage, ice formation, etc. is also available for input via the Bridge/Culvert editor window. Specific instructions on entering bridge and culvert data into HEC RAS is found in Chapter 6 of the HEC RAS User's Manual (2002).

There were a total of 12 bridges and culverts that were included in the model (See Figure 3.1.) Tables A.1 and A.2 contain a list of the bridges and culverts modeled as well as information and dimensions. The information contained in the tables was obtained from the Horry County office of the South Carolina Department of Transportation (Patrick, 2008).

Table A.1. Bridge/culvert dimensions

Roadway	Type	No. of Type	Length (m)	Width (m)	Depth (m)
Long Ave	Bridge	1	25.6	7.8	3.0
Sherwood	Bridge	1	27.4	8.4	3.6
Railriad	Bridge	1	65.5	4.6	3.7
Hwy 701	Box Culvert	1	13.7	24.4	3.6
Millpond Rd 1	Bridge	1	66.4	41.1	3.9
Oak Street	Bridge	1	27.4	14.0	4.1
Millpond Rd 2	Bridge	1	30.2	25.6	3.3
Hwy 501	Box Culvert	1	9.8	28.7	2.4
El Bethel Rd	Bridge	2	32.9	13.1	4.3
Dunn Shortcut	Bridge	1	18.3	10.1	2.9
Sioux Swamp	Bridge	1	13.4	8.22	3.8
Daniel Rd	Round Culvert	2	46.3	1.2	1.2

Table A.2. Bridge/culvert information.

Roadway	No of Piles	No. of Piers	Pier Spacing (m)	No of Channels	Channel Size (m)
Long Ave	35	7	4.3	N/A	N/A
Sherwood	35	7	4.3	N/A	N/A
Railriad	108	18	3.0	N/A	N/A
Hwy 701	N/A	N/A	N/A	4	3.6
Millpond Rd 1	35	2	18.23 21.95 21.64	N/A	N/A
Oak Street	12	2	9.1	N/A	N/A
Millpond Rd 2	20	2	9.1	N/A	N/A
Hwy 501	N/A	N/A	N/A	3	2.4
El Bethel Rd	0	2 (solid)	6.7	N/A	N/A
Dunn Shortcut	30	2	4.6	N/A	N/A
Sioux Swamp	20	2	4.6	N/A	N/A
Daniel Rd	N/A	N/A	N/A	2	N/A

In Table A.1, the depth value is the distance from the bed of the channel up to the bottom of the bridge beams for bridges and up to the interior height of box culverts. The thickness of the bridge deck was not known for all bridges so a thickness of 0.91 m (3 ft) was assumed for all bridges. In order to find the elevation of the top of a bridge deck, the depth reading for that bridge was summed with the assumed thickness of the bridge deck and the elevation of the bottom of the channel where that particular bridge was located. After the dimensions of the bridge deck were defined, the pier dimensions were also defined. Since HEC RAS is not capable of modeling several piles along the length of the pier, a row of piles was input as a single pier. For example: the Long Ave bridge has 35 piles and 7 piers; a row of 5 piles compose a single pier. The deck elevations for culverts were determined using the same methodology as the bridge decks. Instead of inputting pier data, the void area of the culvert was defined as well as information on the inlets and outlets of the culverts. Box culverts at Hwy 701 and Hwy 501are 0.3m (1ft) and 0.45m (1.5ft) above the downstream channel elevation respectively. The sudden elevation change of channel thalweg at these culverts limit downstream tidal backwater flows from affecting upstream reaches.

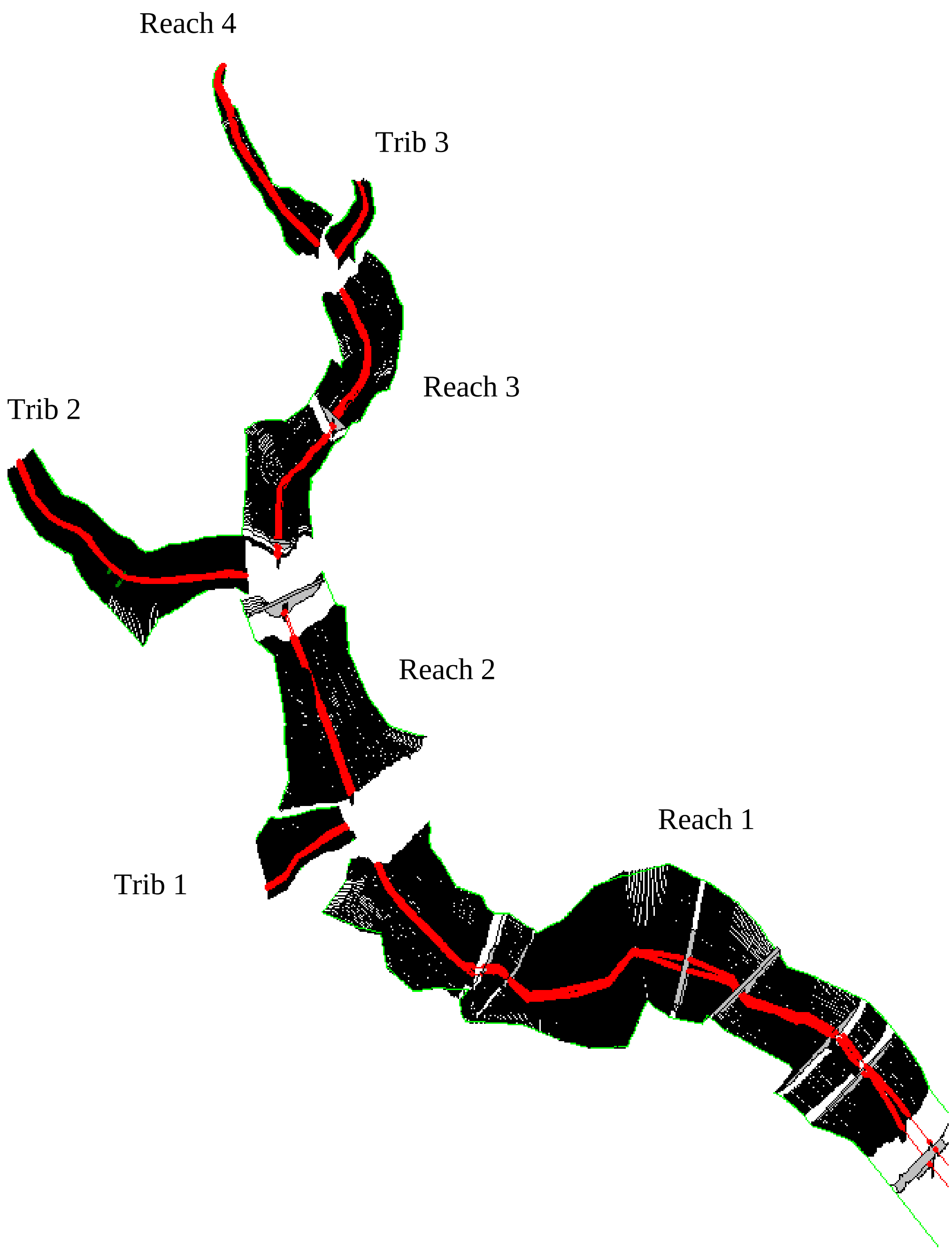


Figure A.1. 3D view of model in HEC RAS.

Appendix B

Performing Unsteady State Flow Analysis

The required input information for an unsteady flow analysis include the boundary conditions at all external boundaries of the system and initial flow and storage area conditions at the beginning of simulation. Upstream boundary conditions, which were located at the top of upstream reaches and at the uppermost points of the modeled tributaries, were modeled using flow hydrographs. The only downstream boundary condition was at the downstream end of Reach 1. A normal depth boundary condition was used as the downstream boundary condition.

When plans were created for the unsteady flow simulations, all three programs were selected to run: Geometry Preprocessor, Unsteady Flow Simulations and Post Processor. Unsteady simulations were ran for 6 days, with a starting time of 0100 on the first day and an ending time of 2340 on the sixth day. A computational interval of 30 seconds was used with a Hydrograph Output interval of 1 minute and a Detailed Output Interval of 10 minutes. Rainstorm duration was less than 1 day but a 6 day window was used to run the simulations to ensure leveling out of hydrograph. Data points were taken at maximum water surface for each cross section so therefore the time at which the points were taken during the simulation was not of concern.

Appendix C

Development of Win TR-55 Model

For each reach, the required input data included the receiving reach, reach length, a Manning n value, friction slope, bottom width, and average side slopes. Table C.1 contains all of the reach data used.

Table C.1. Reach data used.

Reach Name	**Receiving Reach**	**Reach Length (m)**	**Manning n**	**Friction Slope (m/m)**	**Bottom Width (m)**	**Average Side Slopes**
Reach 1	Outlet	4251	0.035	0.001	13.7	1:1
Trib 1	Reach 1	648	0.035	0.002	1.8	1:1
Reach 2	Reach 1	1131	0.035	0.001	7.6	1:1
Trib 2	Reach 3	1757	0.035	0.002	3.0	1:1
Reach 3	Reach 3	1728	0.035	0.001	3.0	1:1
Trib 3	Reach 4	448	0.035	0.002	1.8	1:1
Reach 4	Reach 4	1198	0.035	0.003	1.8	1:1

The second step in data input into WinTR-55 is the input of land use details. Land used details and Hydrologic Soil Group types were used to obtain curve number (CN) values for each subwatershed. A curve number is a number that is used to indicate the stormwater runoff potential of a given area of land and is dependent on the land use description and the hydrologic soil grouping of the given soil (Haan, 1994). Land use details are described in Tables C.2 and C.3.

Table C.2. Description of land covers used in Win TR-55 and an assigned ID number.

Cover Description	**Identification Number**
Fully developed urban area with established vegetation; fair condition with 50-75% grass cover	1
Fully developed urban area with established vegetation; poor condition with less than 50% grass cover	2
Brush, weed, grass mix in fair condition	3
Woods in fair condition	4
Straight row crop with crop residue in good condition	5

Table C.3. Land use details.

Subarea Name	**Cover ID No.**	**Area (km²) for Hydrologic Soil Group (HSG)**							
		HSG A	**CN**	**HSG B**	**CN**	**HSG C**	**CN**	**HSG D**	**CN**
Reach 4	1	0.409	49	0.047	69	0.104	79	0.030	84
Reach 4	3	1.595	35	0.179	56	0.401	70	1.189	77
Reach 4	4	0.583	36	0.065	60	0.148	73	0.433	79
Trib 3	1	0.044	49	0.026	69	0.085	79	0.101	84
Trib 3	3	0.122	35	0.067	56	0.238	70	0.277	77
Trib 3	4	0.184	36	0.104	60	0.357	73	0.420	79
Trib 2	1	0.000	-	0.174	69	0.228	79	0.723	84
Trib 2	2	0.000	-	0.016	79	0.021	86	0.067	89
Trib 2	3	0.000	-	0.578	56	0.813	70	2.564	77
Trib 2	4	0.000	-	1.075	60	1.515	73	4.776	79
Trib 2	5	0.000	-	0.433	75	0.611	82	1.927	85
Trib 1	1	0.119	49	0.215	69	0.101	79	0.490	84
Trib 1	4	0.655	36	1.176	60	0.552	73	2.668	79
Trib 1	5	0.096	64	0.171	75	0.080	82	0.389	85

Table C.4 Weighted curve numbers for each subarea.

Sub-area Name	**Weighted Curve Number**
Reach 4	57
Tributary 3	68
Tributary 2	76
Tributary 1	71

Time of concentration details are also taken into account for each subarea. The length was approximated by using the measuring tool in ArcMap™. The average slope was found by using a DEM to obtain approximate elevation drops across subareas and dividing the elevation drop by the length of the subarea. The surface is an average of the entire subarea. See Table C.5 for the time of concentration details.

Table C.5. Time of concentration details.

Reach 4								
Flow Type	**Length (m)**	**Slope**	**Surface**	**n**	**Area (m²)**	**WP (m)**	**Velocity (m/s)**	**Time (s)**
Sheet	30	0.003	Light Woods	-	-	-	-	2462
Shallow Concentrated	884	0.003	Unpaved	-	-	-	-	3283
Channel	2438	0.004	-	0.035	0.2	0.6	0.8	2970
Total	**3353**							**8716**
Tributary 3								
Flow Type	**Length (m)**	**Slope**	**Surface**	**n**	**Area (m²)**	**WP (m)**	**Velocity (m/s)**	**Time (s)**
Sheet	30	0.0024	Light Woods	-	-	-	-	2693
Shallow Concentrated	1219	0.0024	Unpaved	-	-	-	-	5062
Channel	732	0.0021	-	0.035	0.3	0.8	0.7	1091
Total	**1981**							**8845**
Tributary 2								
Flow Type	**Length (m)**	**Slope**	**Surface**	**n**	**Area (m²)**	**WP (m)**	**Velocity (m/s)**	**Time (s)**
Sheet	30	0.0019	Light Woods	-	-	-	-	2956
Shallow Concentrated	305	0.0019	Unpaved	-	-	-	-	1422
Channel	1829	0.0021	-	0.035	0.5	2	0.6	3074
Total	**2164**							**7452**
Tributary 1								
Flow Type	**Length (m)**	**Slope**	**Surface**	**n**	**Area (m²)**	**WP (m)**	**Velocity (m/s)**	**Time (s)**
Sheet	30	0.002	Light Woods	-	-	-	-	2894
Shallow Concentrated	914	0.002	Unpaved	-	-	-	-	4158
Channel	3414	0.002	-	0.035	0.9	2	0.9	3704
Total	**4359**							**10757**

Appendix D

Hydrographs

Hydrographs were generated in WinTR-55 to be used as unsteady flow data within HEC RAS. Initially 2-year hydrographs were generated and used. Stability issues arose when running unsteady flow simulations. It was predicted that model instabilities were due to the fact that not enough flow was coming through the channel. A multiplier was applied to the hydrographs being used so that they corresponded to a storm larger than a 2-year storm but smaller than a 5-year storm. A minimum flow amount was also factored into the input hydrograph. A multiplier of 7 was used for all hydrographs. A minimum discharge of 0.14 m^3/s (5 cfs) was applied to Tributaries 1 and 2. A minimum discharge of 0.06 m^3/s (2 cfs) was applied to Tributary 3 and a minimum discharge of 0.17 (6 cfs) was applied to Reach 4. Both the multiplier tool and minimum flow amount tool are options available within HEC RAS. The hydrographs used produced a flow depth greater than or equal to 0.15 m (6 in) on the widest floodplain (FPR 20). Figures D.1, D.2, D.3 and D.4 show the hydrographs with a multiplier and a minimum discharge. Table D.1 contains the peak flow value for each hydrograph.

Table D.1. Peak flows of hypothetical storm used.

Reach	**Peak Flow (m^3/s)**
Tributary 1	1.66
Tributary 2	6.62
Tributary 3	0.87
Reach 4	6.62

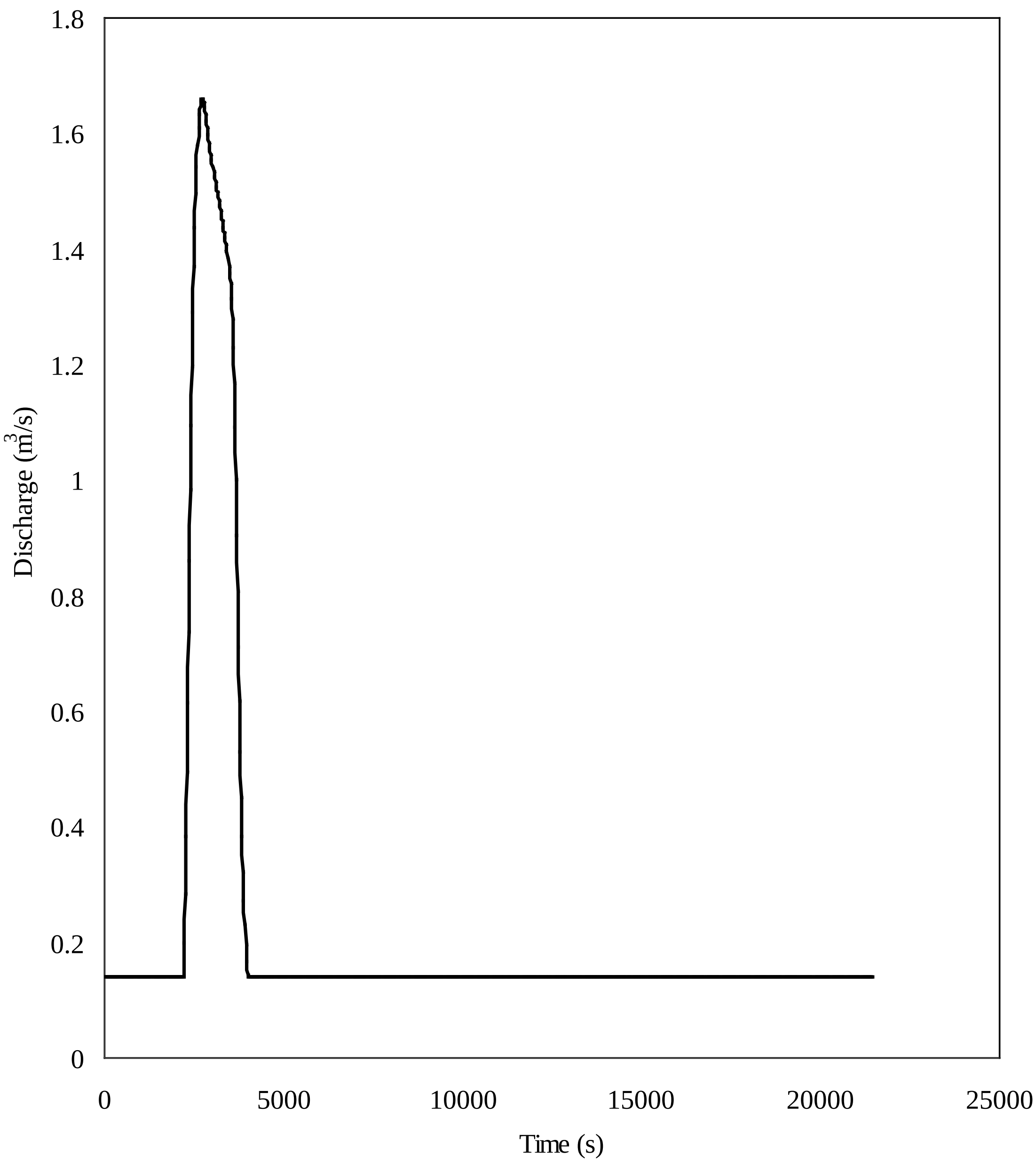


Figure D.1. Hydrograph applied at top if Tributary 1.

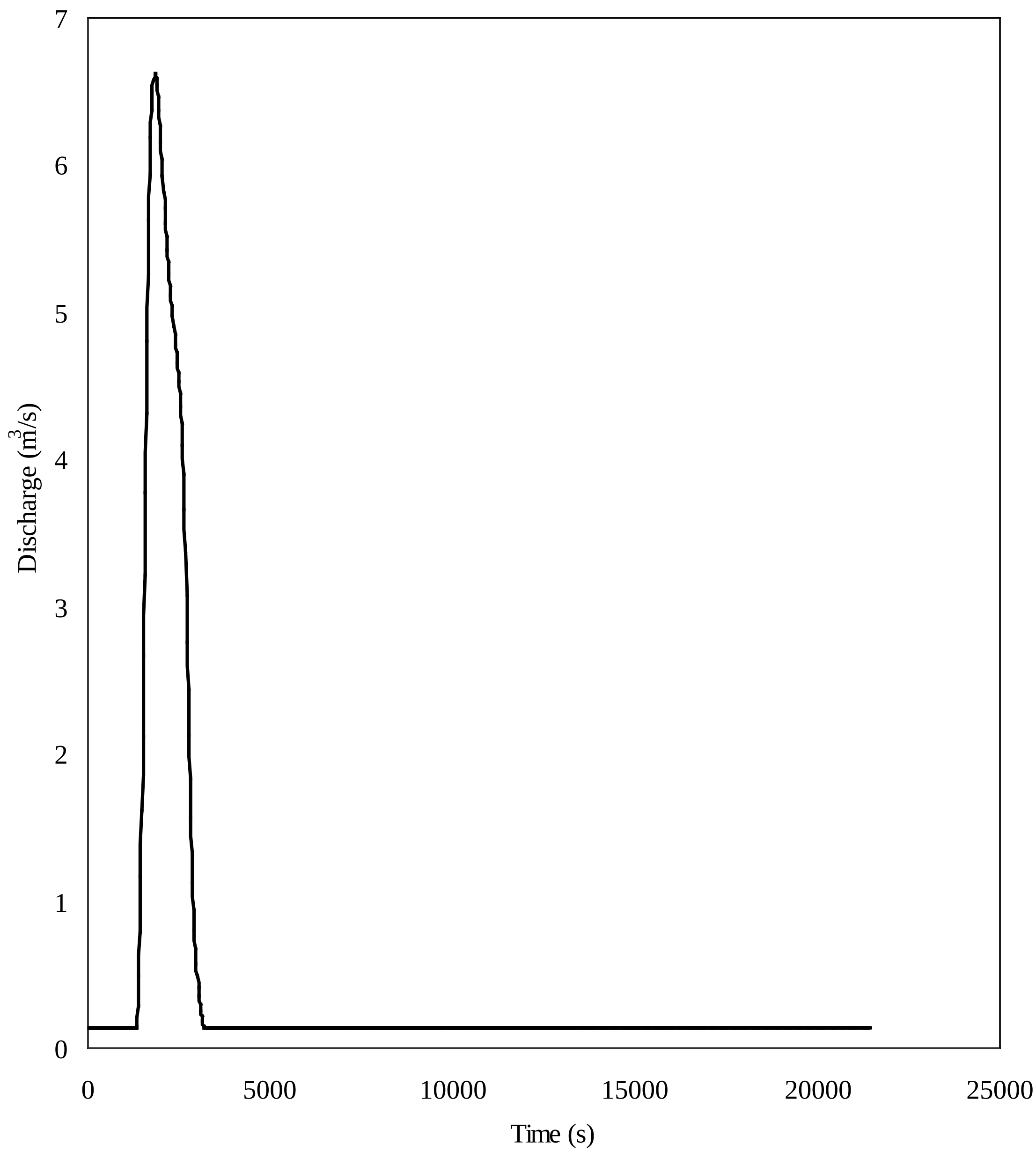


Figure D.2. Hydrograph applied at top of Tributary 2.

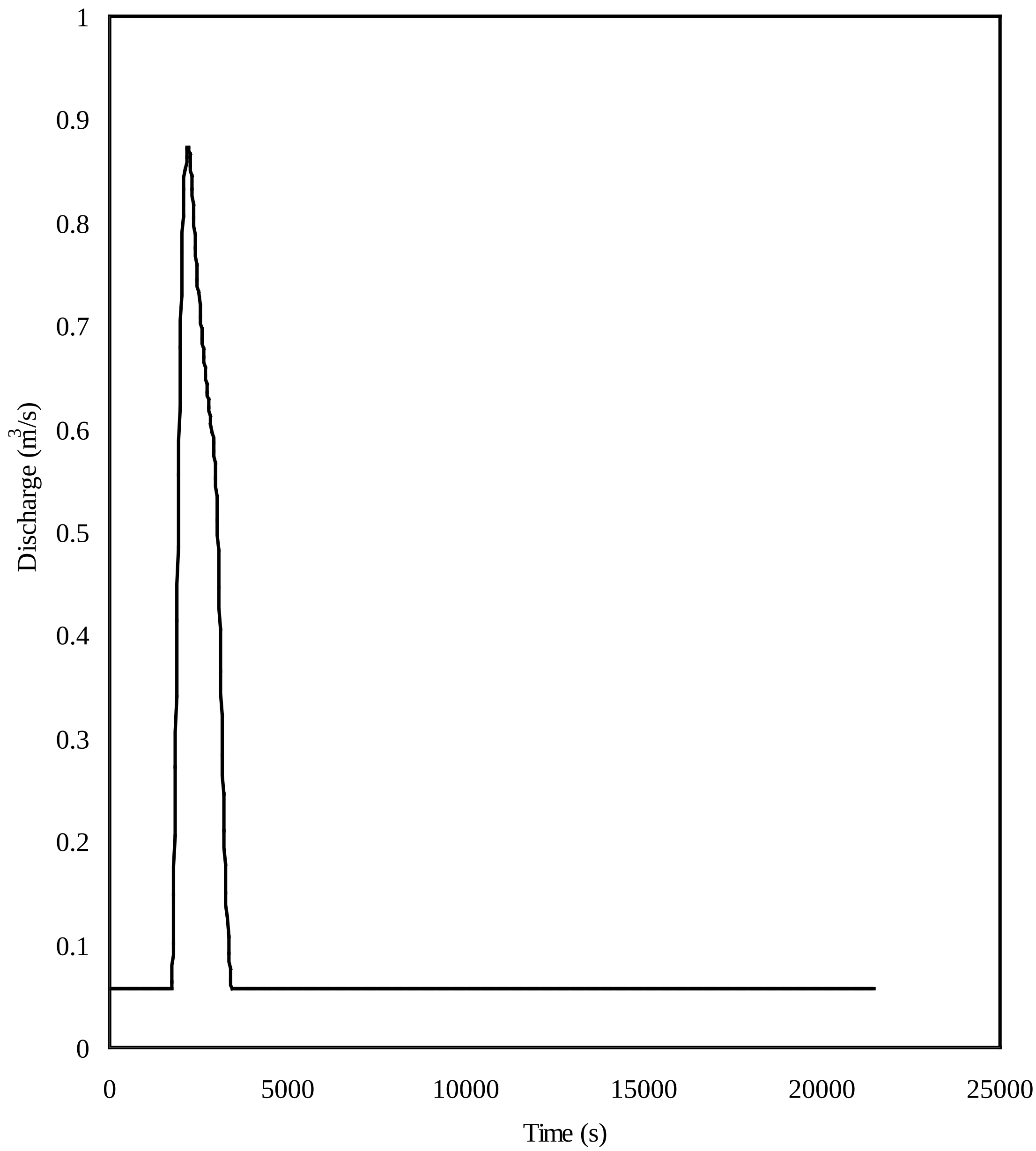


Figure D.3. Hydrograph applied at top of Tributary 3.

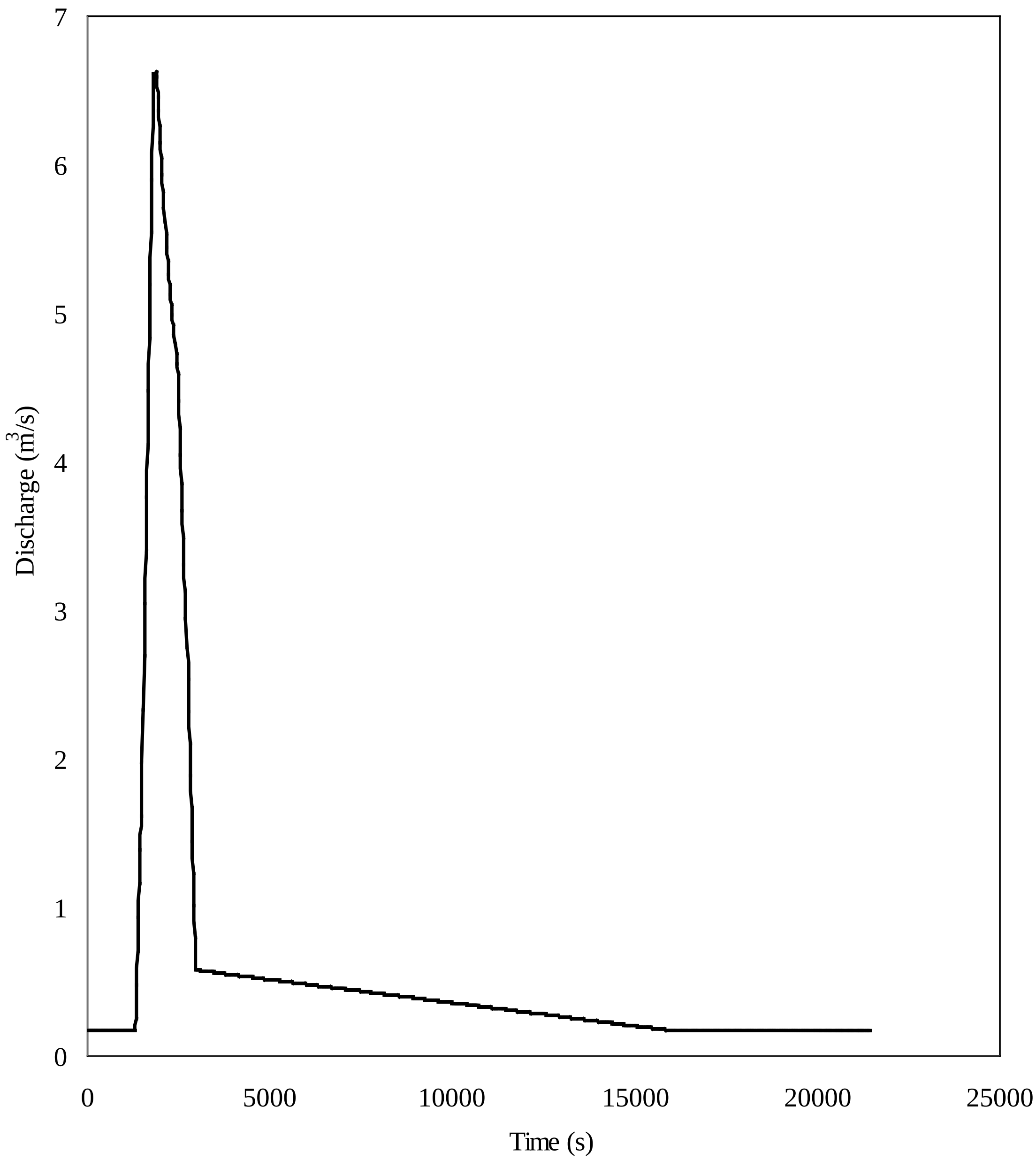


Figure D.4. Hydrograph applied at top of Reach 4.

Appendix E

Determination of Critical Shear Stress

Web Soil Survey (WSS) (websoilsurvey.nrcs.usda.gov) was used to determine what type of soil is present in areas immediately surrounding the channel itself (an approximate 6 m (20 ft) buffer was used). The predominate soil type present is a Meggett loam. According to the National Cooperative Soil Survey (NCSS) a Meggett loam can be slightly stick to very sticky (cohesive) and has a moderate shrink-swell potential (USDA-NRCS, 2008a). The porosity of a Meggett loam is 42.2% (Peele et al., 1970). Equation 5 (Das, 2006) was used to find the void ratio of the soil. The void ratio was then used in conjunction with Figure E.1 to determine a unit tractive force or critical shear stress for Crabtree Canal. The curves in Figure E.1 are converted from USSR (1936) permissible velocity data from straight channels with an average depth of 0.91 m (3 ft) (USDA-NRCS, 2007).

$$e = \frac{n}{1-n} \tag{5}$$

Where: n=porosity and

e=void ratio.

If the porosity of a Meggett loam is 42.2%, then the void ratio is calculated as:

$$e = \frac{n}{1-n}$$
$$e = \frac{0.422}{1-0.422}$$
$$e = 0.851$$

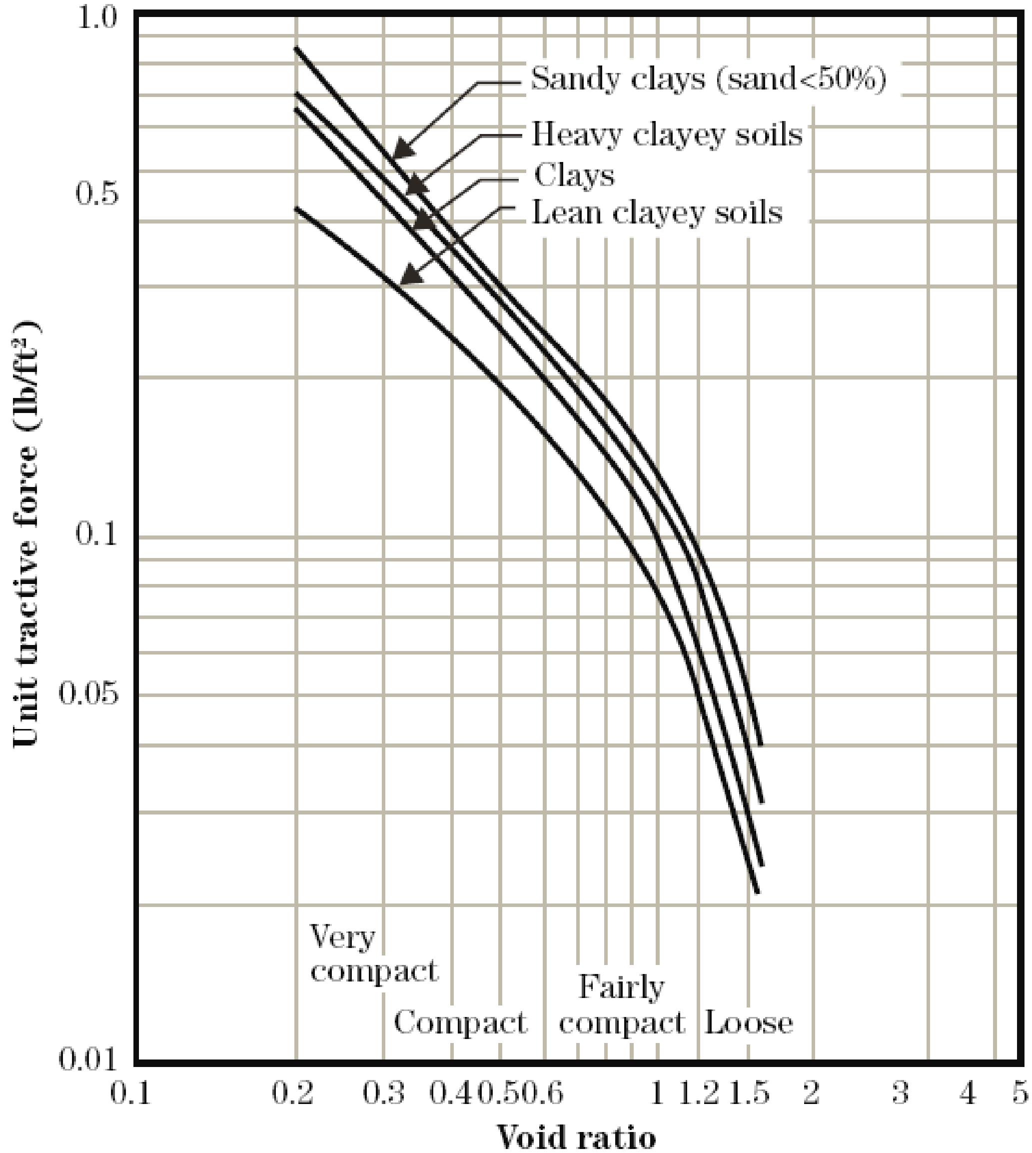


Figure E.1. Allowable shear stress in cohesive material in straight trapezoidal channels (USDA-NRCS, 2007).

From WSS, a Meggett loam is 42.1% sand, therefore the curve for a sandy clay was used to deteremine that a Meggett loam has a critical shear stress of approximatly 7.7 N/m² (1.6 lb/ft²). This value was within 0.5 N/m² to the tractive force value given by Lane (1955). According to Lane (1955) an ordinary frim loam has a travtive force value of 7.2 N/m² (1.5 lb/ft²). The two values were very similar, but the smaller value of 7.2 N/m² was used to compare with HEC RAS predictions.

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